Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 NOVEMBER 1959

DEC 29 1959
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COLLEGE STATION, TEXAS

TO DOUBLE THE POWER OF ROXBURGH

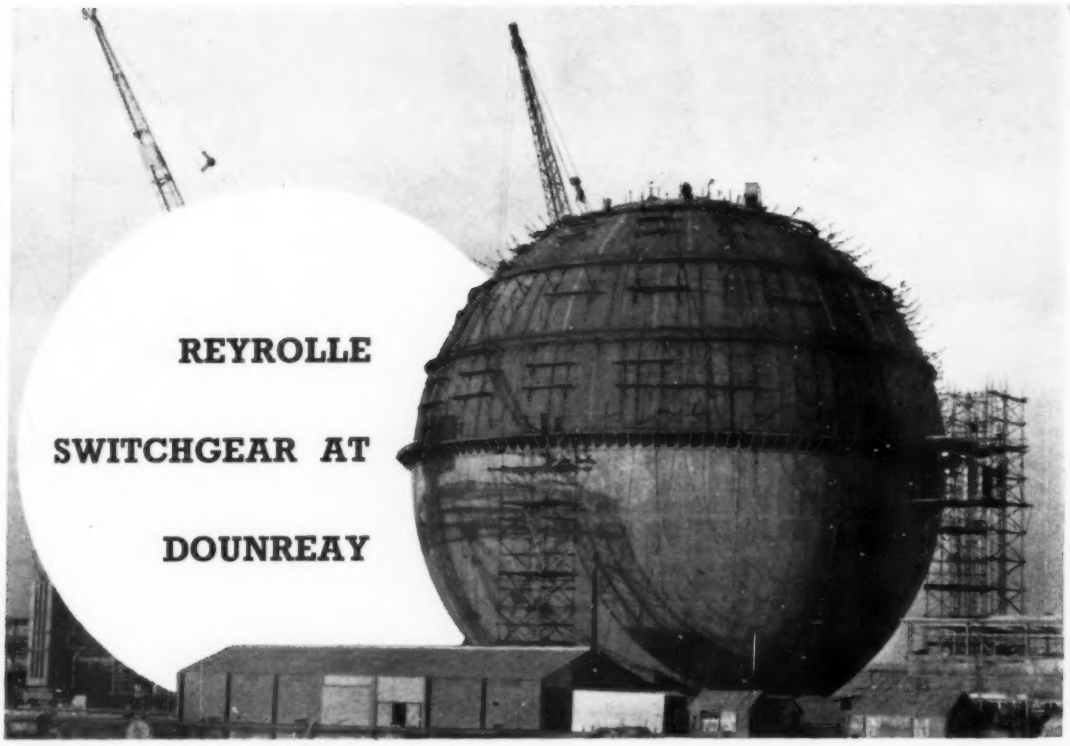
AEI

Four AEI waterwheel alternators at the New Zealand Electricity Department's Roxburgh Power Station are already feeding 160,000 kW into the South Island grid. Four additional sets, to bring the station up to its designed capacity of 320,000 kW, are now in course of construction.

ASSOCIATED ELECTRICAL INDUSTRIES LIMITED

HEAVY PLANT DIVISION RUGBY AND MANCHESTER, ENGLAND

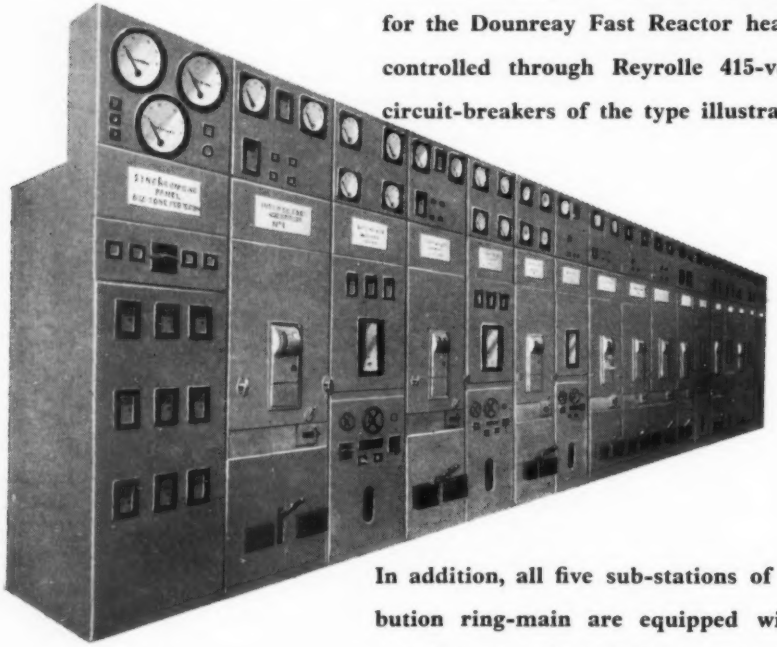
AS217



REYROLLE SWITCHGEAR AT DOUNREAY

Photograph by courtesy of the U.K.A.E.A

The electro-magnetic liquid-metal pumps and other services for the Dounreay Fast Reactor heat-exchanger system are controlled through Reyrolle 415-volt 31·5-MVA air-break circuit-breakers of the type illustrated.



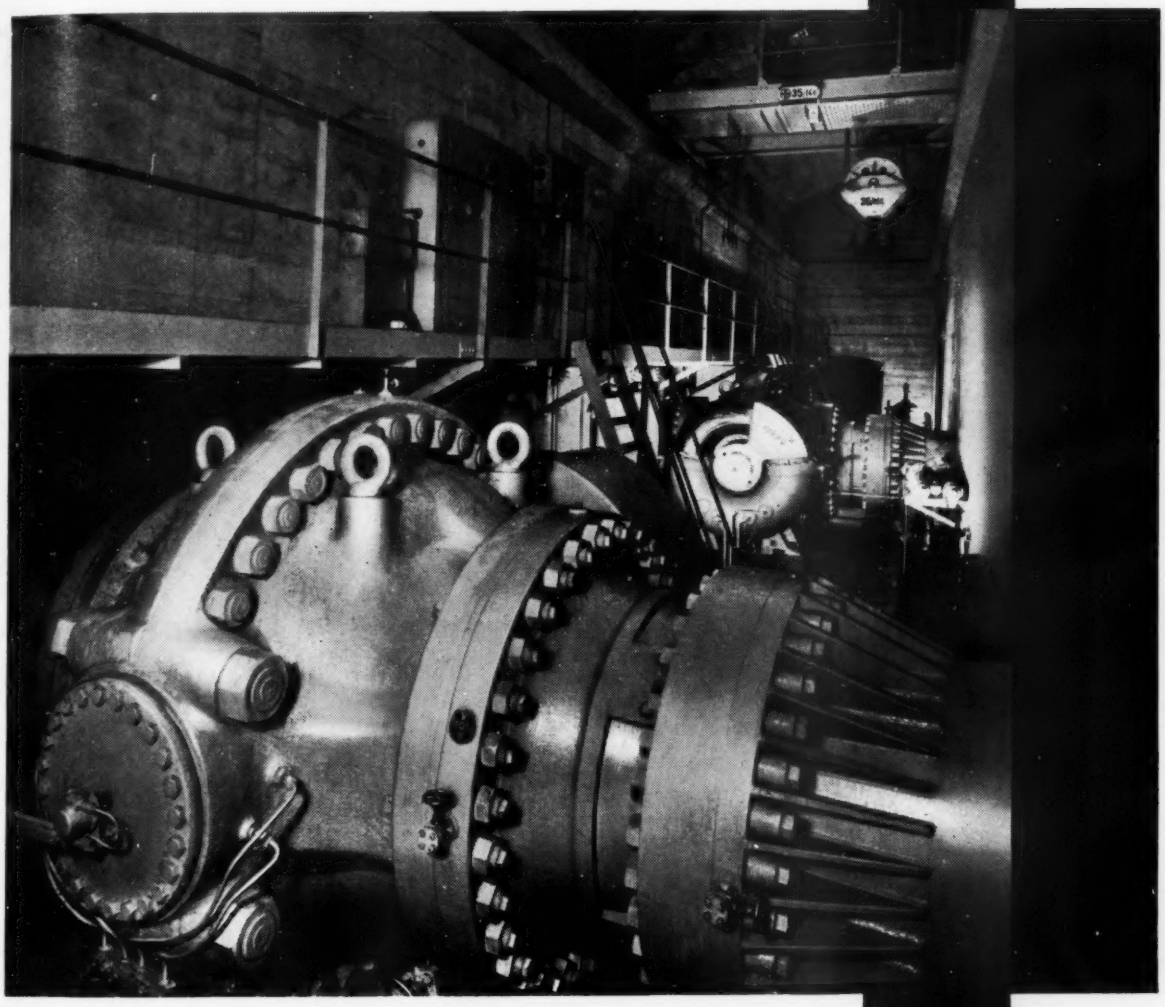
17-panel switchboard comprising Reyrolle type-S22A air-break circuit-breakers with associated control cubicles.

In addition, all five sub-stations of the main 11-kV distribution ring-main are equipped with Reyrolle metalclad switchboards with 350-MVA horizontal draw-out oil-break circuit-breakers.

Reyrolle

A. Reyrolle & Co. Ltd · Hebburn · County Durham · England

Equipment for Hydro-Electric Power Stations



Valve chamber with rotary slide sphériques valves as turbine shut-off device in a swiss power station

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76

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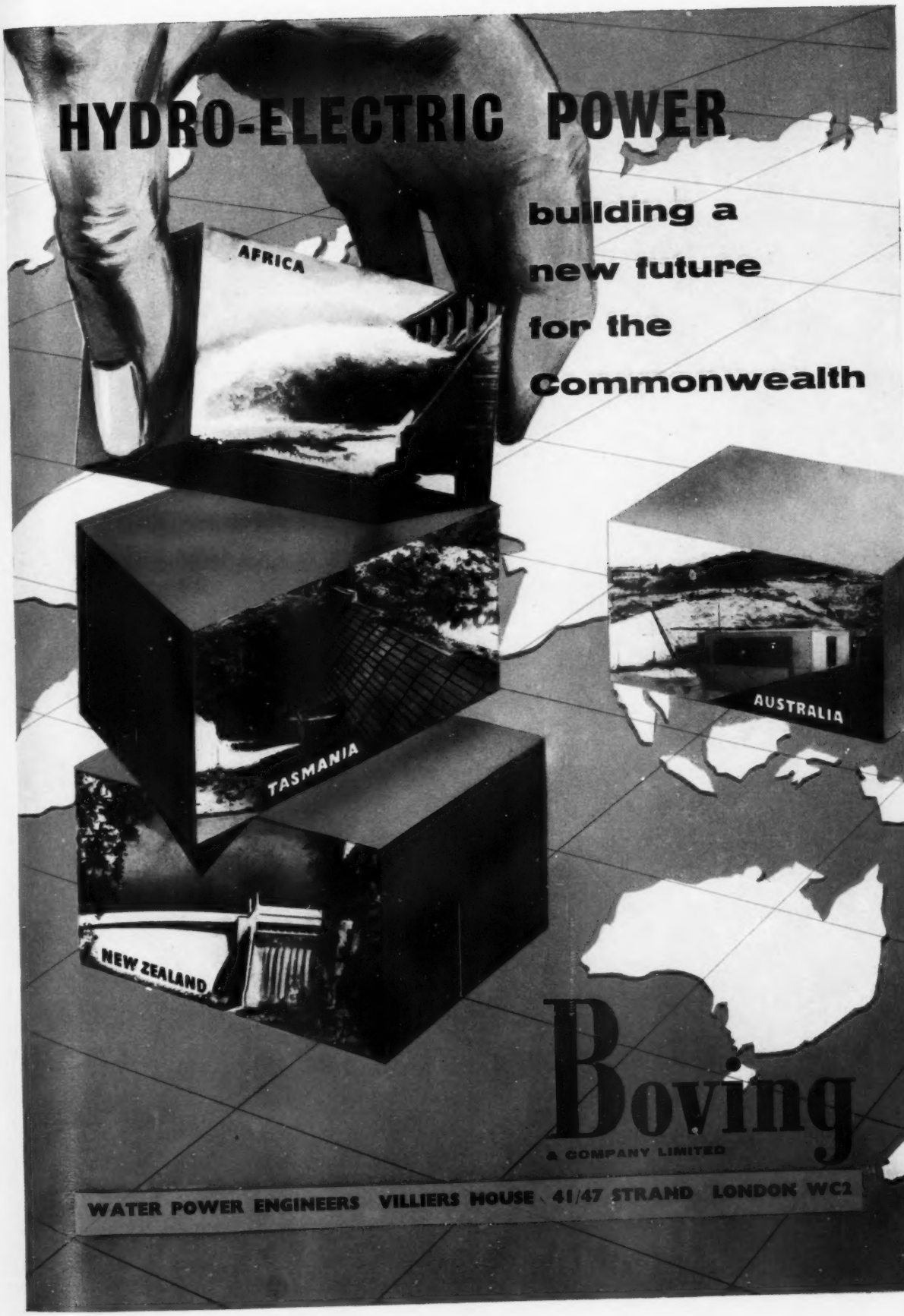
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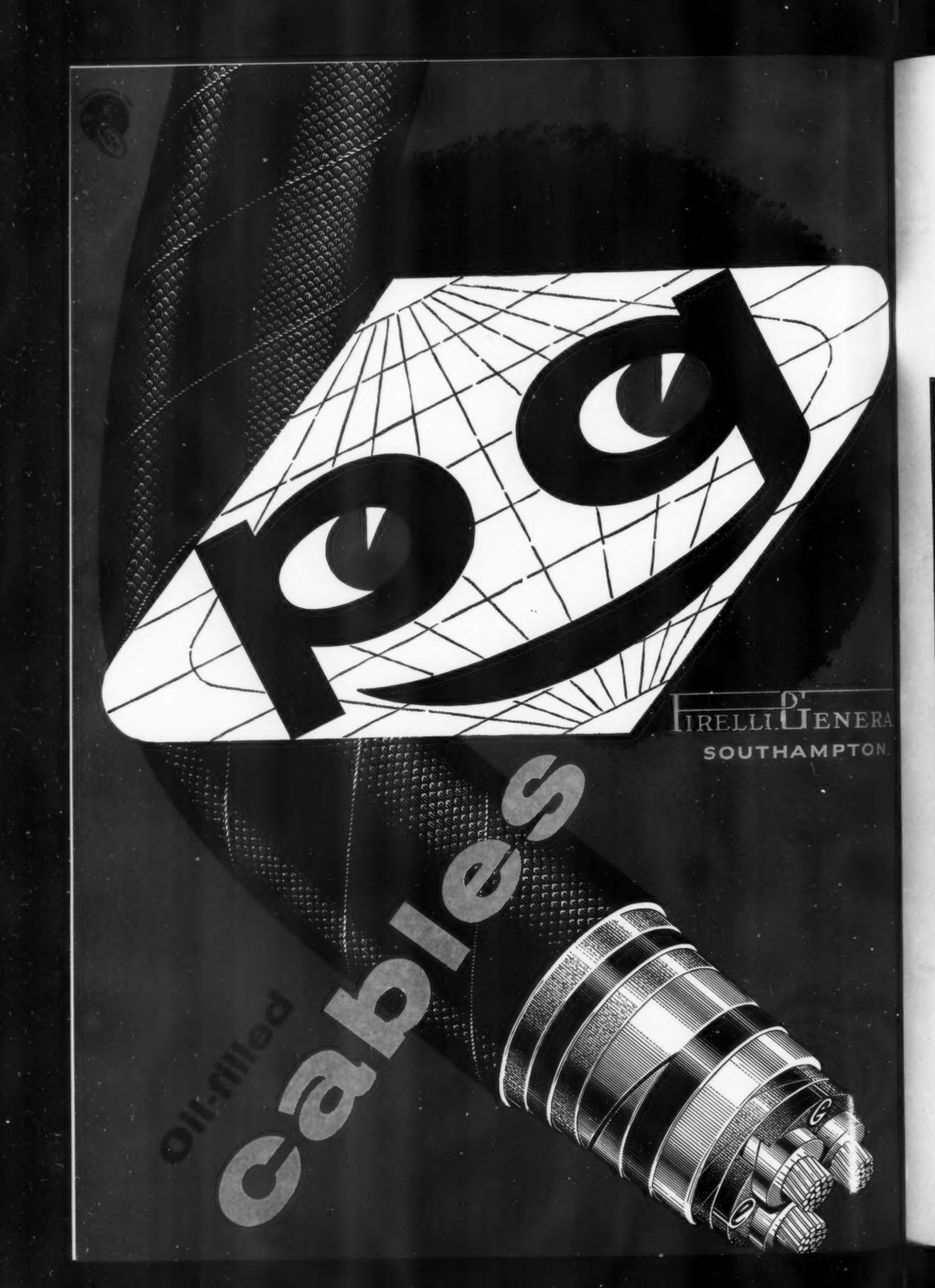
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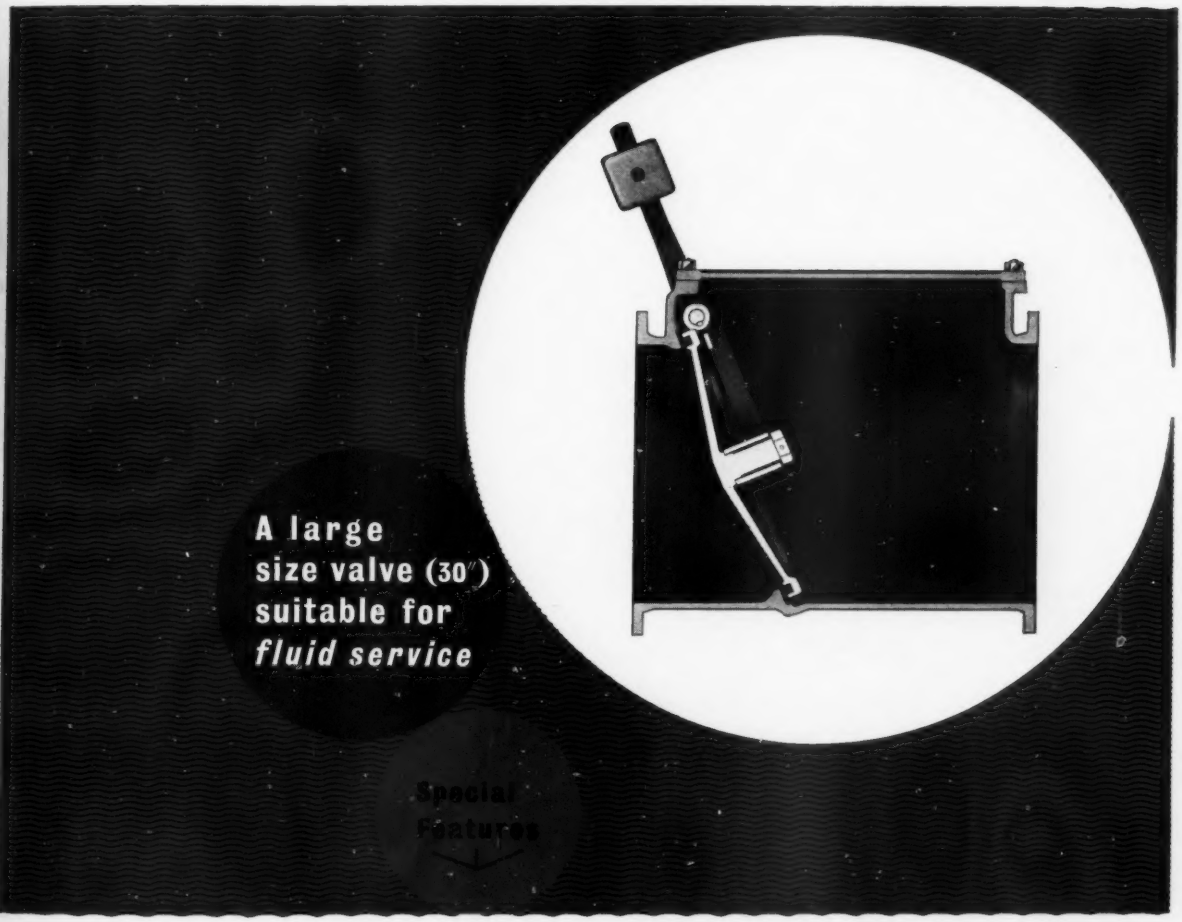

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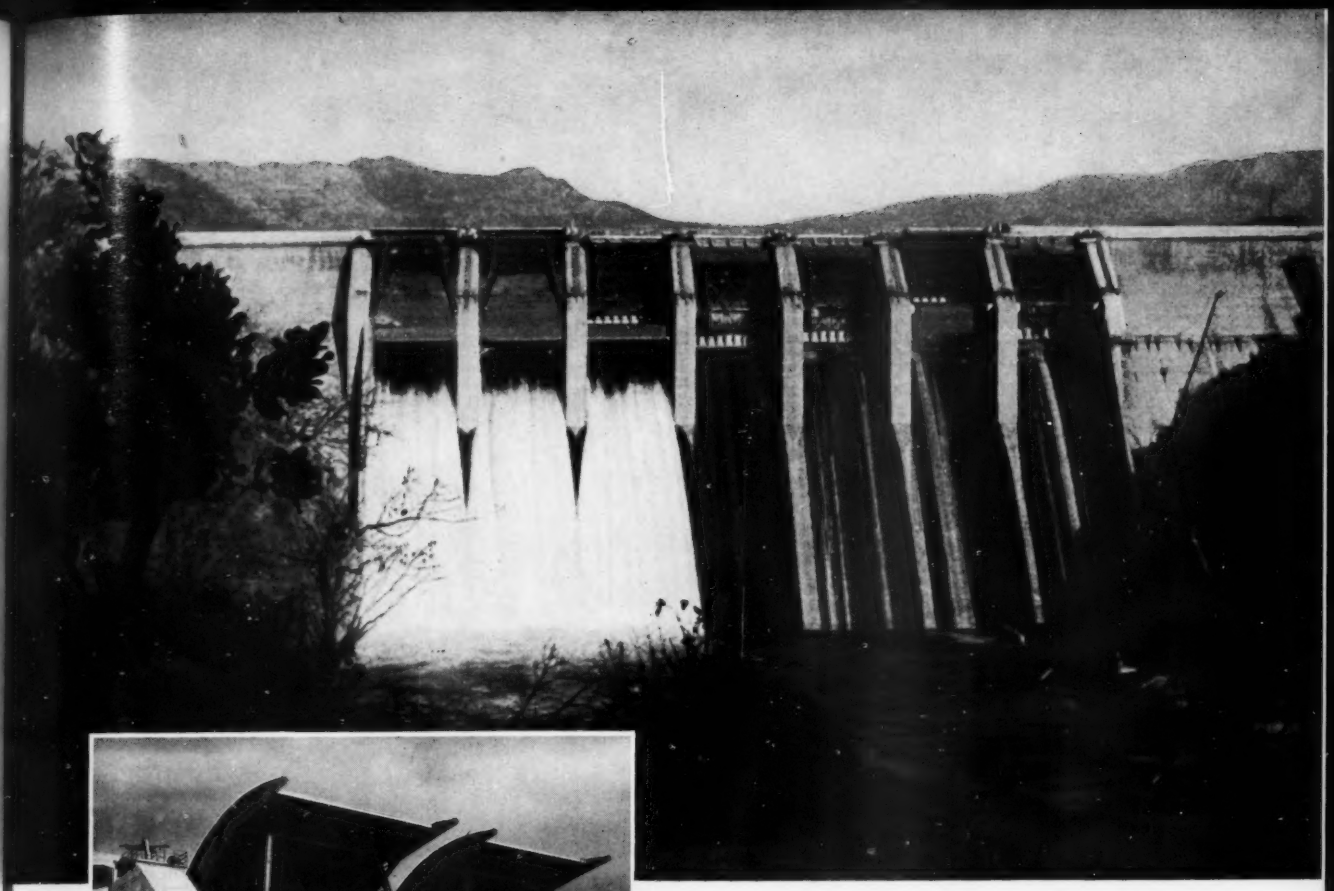

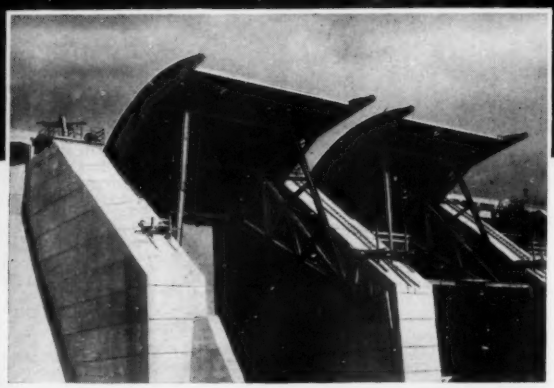

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Photos 1 and 2
7 spillway tainter gates 12 x 12 m for dam flood control

Photo 3
Wye pipe for hydraulic power conduit
Inlet opening 7010 mm ø
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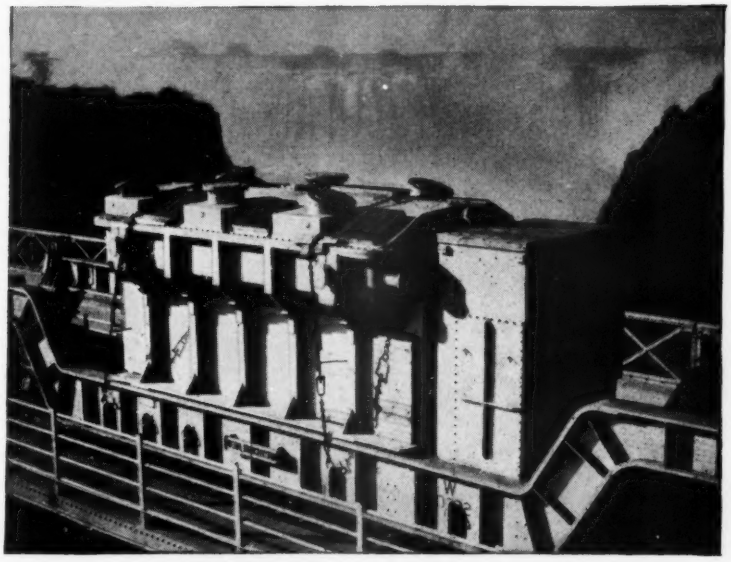
The consulting electrical and mechanical engineers are Messrs. Merz & McLellan.



A Ferranti large power transformer crossing Victoria Falls bridge on its way to the Kitwe sub-station for the Kariba Hydro-Electric scheme. Two of these transformers on special well-wagons form part of the load on a goods train.

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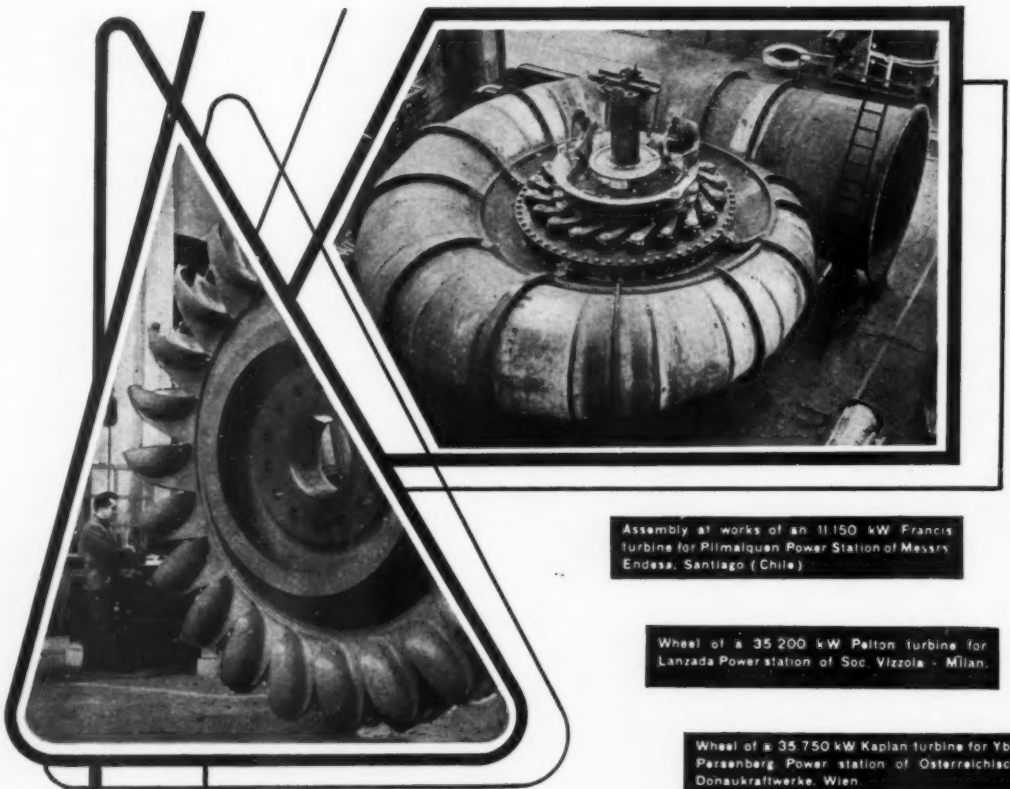
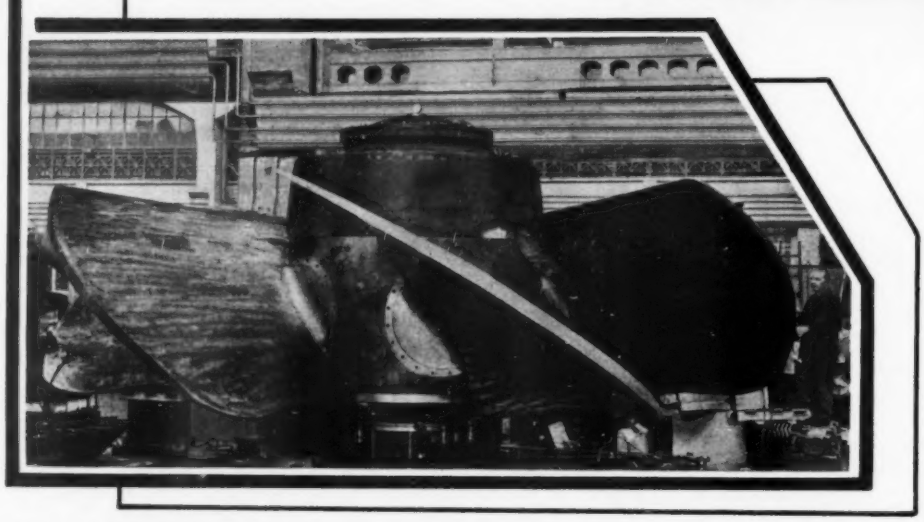


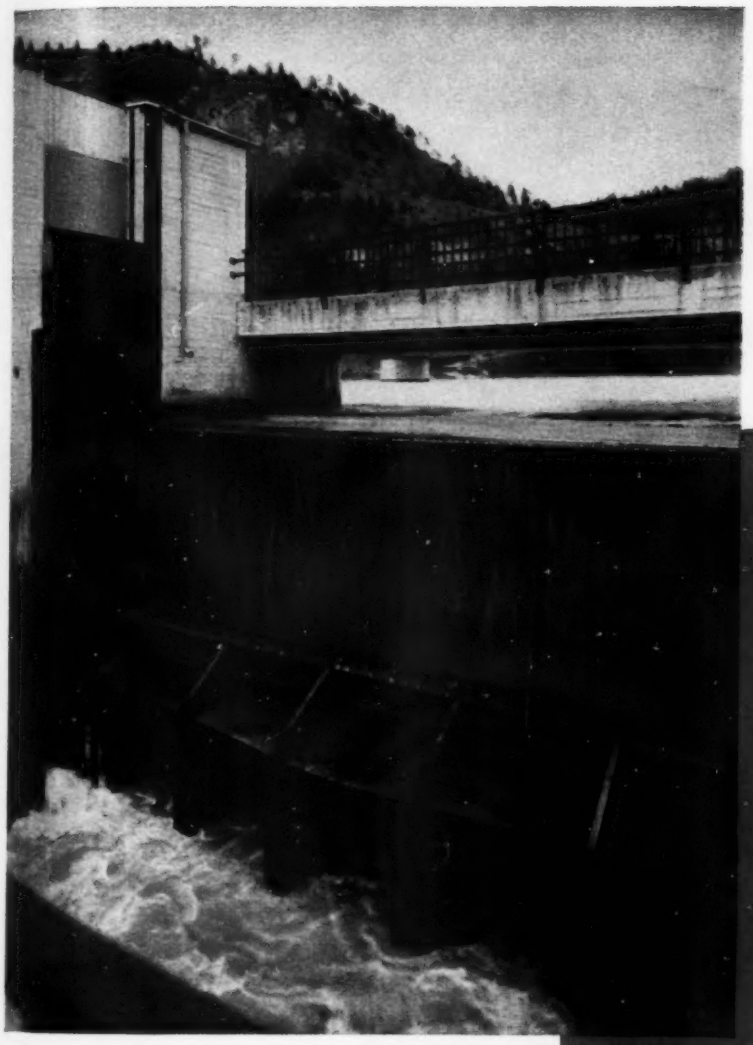
Assembly at works of an 11.150 kW Francis turbine for Pilmaiquen Power Station of Messrs Endesa, Santiago (Chile).

Wheel of a 35.200 kW Pelton turbine for Lanzada Power station of Soc. Vizzola - Milan.

Wheel of a 35.750 kW Kaplan turbine for Ybbs Persenberg Power station of Osterreichische Donaukraftwerke, Wien.



ANSALDO-SAN GIORGIO
GENOVA



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M·A·N hook-type gates enable, besides fine regulation by lowering the top gate by approx. 40% of the total retaining height, the discharge of average floodwater without operating the bottom gate. Füssen/Lech Weir Plant with 3 M·A·N hook-type gates of 10.5 m clear width and 9.0 m retaining height each.

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are preferred for major jobs

On public works construction as well as quarrying and mining, Bucyrus-Erie heavy-duty shovels and draglines are preferred. Illustrated is a 110-B 4¼ cubic yard shovel, one of three employed on the Serre-Poncon earth dam project on the Durance river in Southern France. Two 150-B draglines were engaged on the same scheme. Over a hundred years have gone by since dam projects in this region were first visualised. Only the development of present-day technical resources has made the project possible—and the choice of excavators? Bucyrus-Erie.

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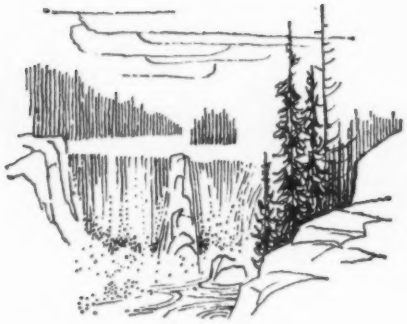

water power

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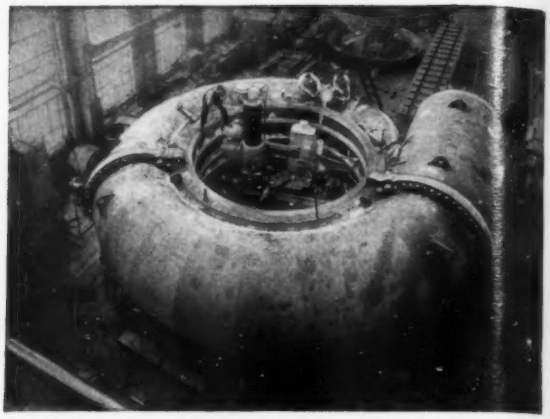
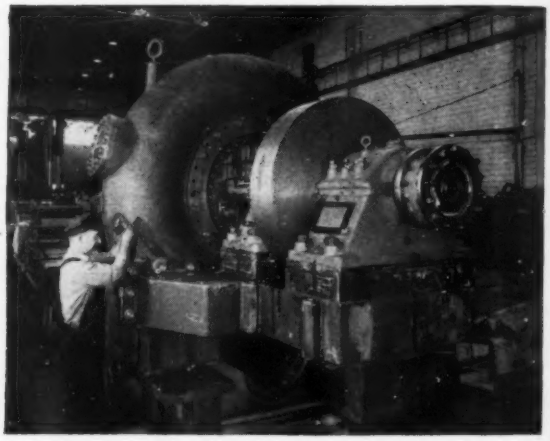
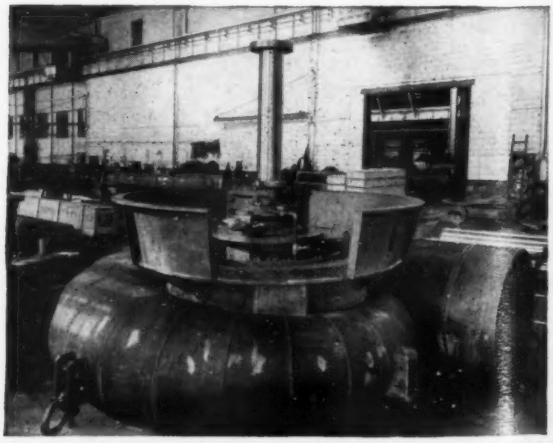


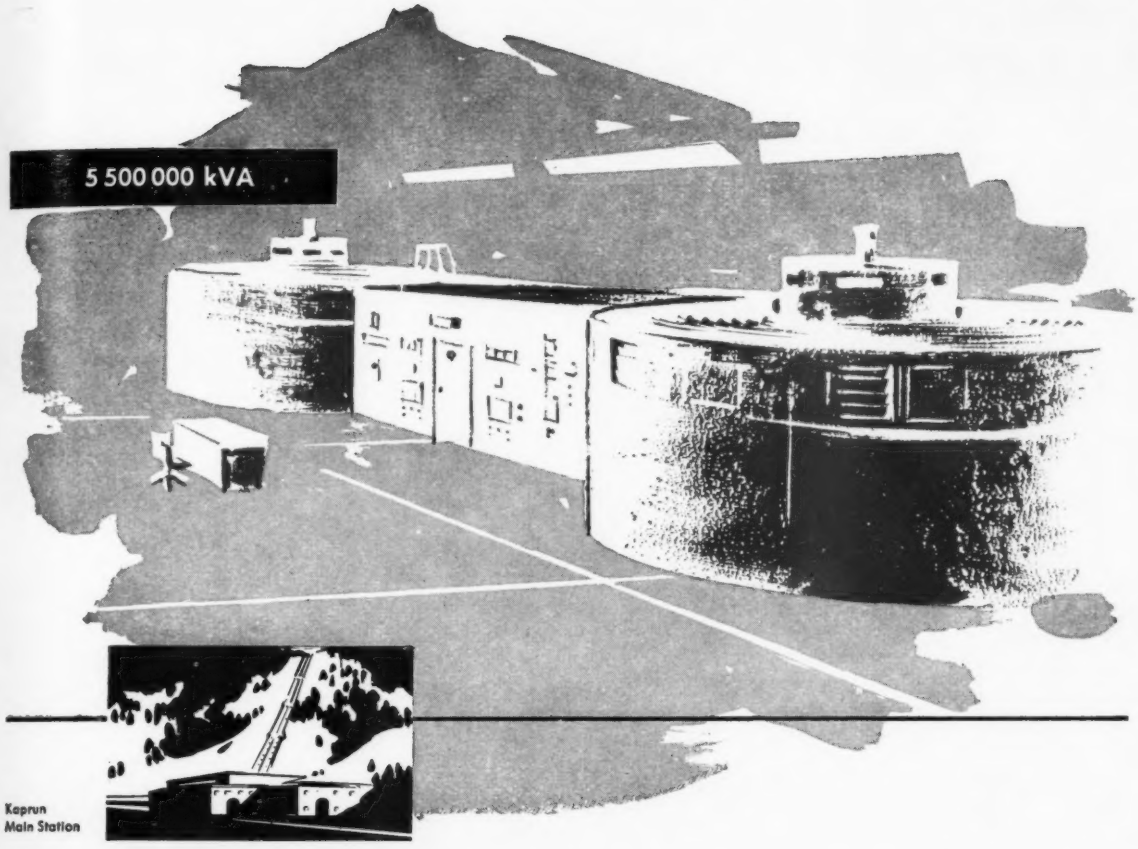
Scroll Case for 150,000 H.P. Neyrpic Turbine working under 785′ Net head, being machined in our shops.



A Horizontal Shaft Leffel Turbine rated 3,200 H.P. under 425′ Net head, partly assembled.

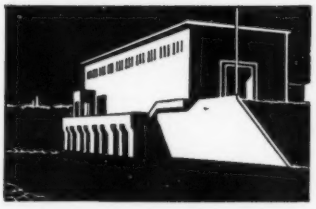


Shop assembly of Vertical Leffel Turbine rated 3,000 H.P. under 125′ Net head.

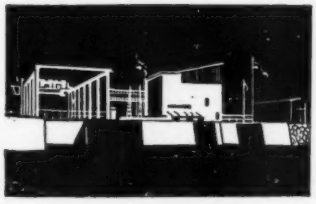


Kaprun
Main Station

Sariyar
Hydro-Electric
Power Station



Pamila
Hydro-Electric
Power Station



5 500 000 kVA
is the aggregate rating of water wheel generators AEG so far supplied or for which contracts have been placed.

Since the beginning of 1946 alone contracts for the supply of water wheel generators from smallest ratings up to 100000 kVA units totalling over 2000000 kVA have been placed with our organization.

During the past 60 years AEG have built plants for generation, transmission and distribution of electrical energy obtained from hydro-electric stations and billions of kWh are made available anually by AEG water wheel generators.

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Tavanasa	**480**	**67 000**
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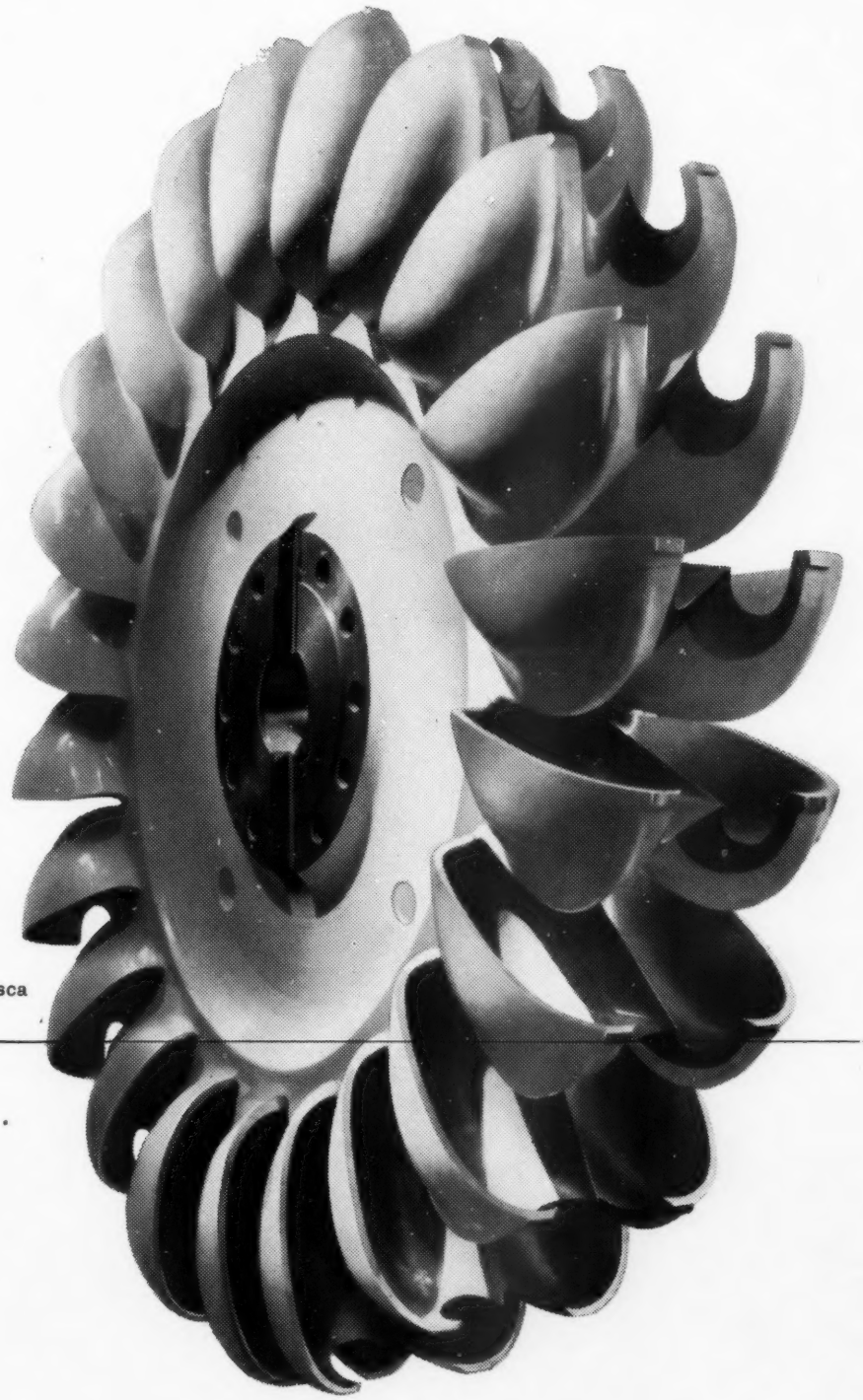
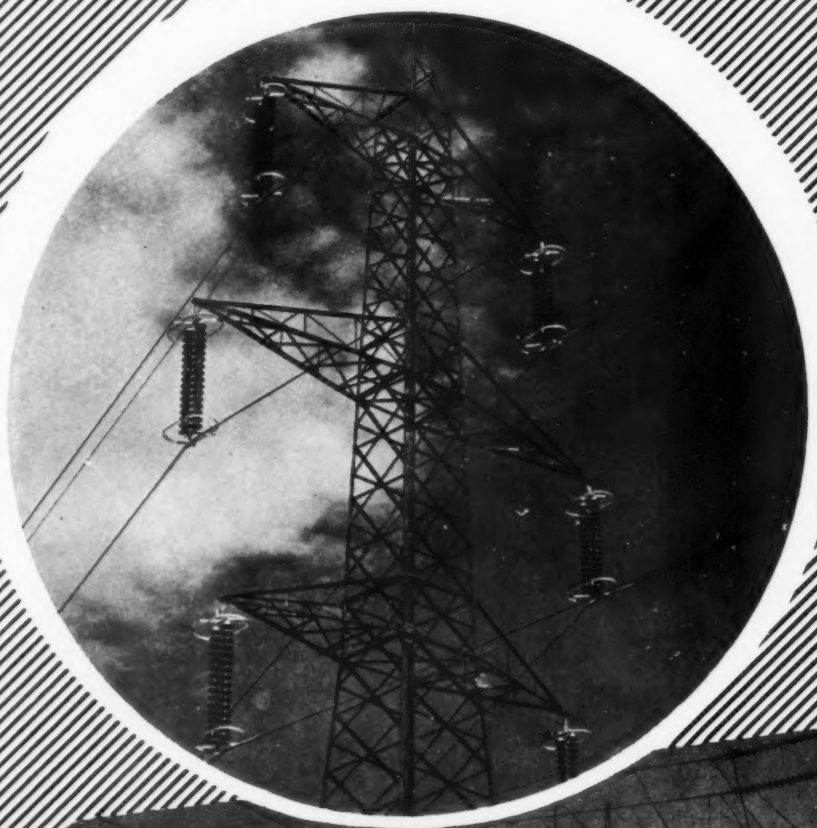
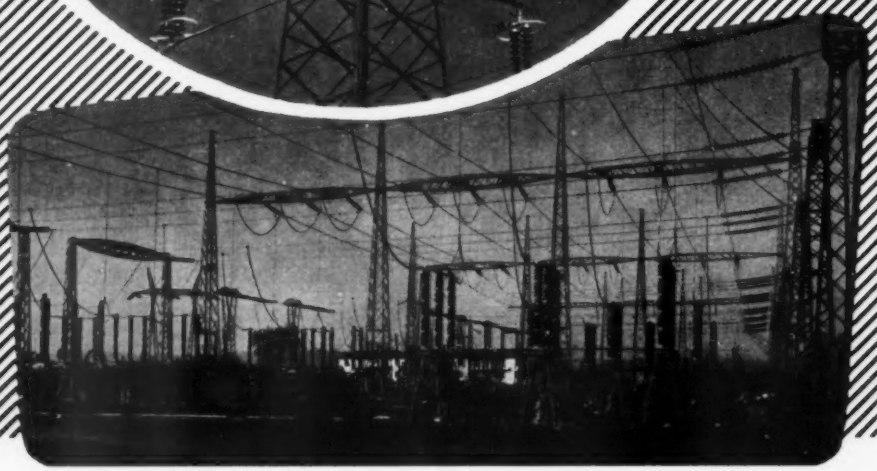
Escher Wyss Ltd., Zurich/Switzerland



One of the runners for Biasca

SALVI





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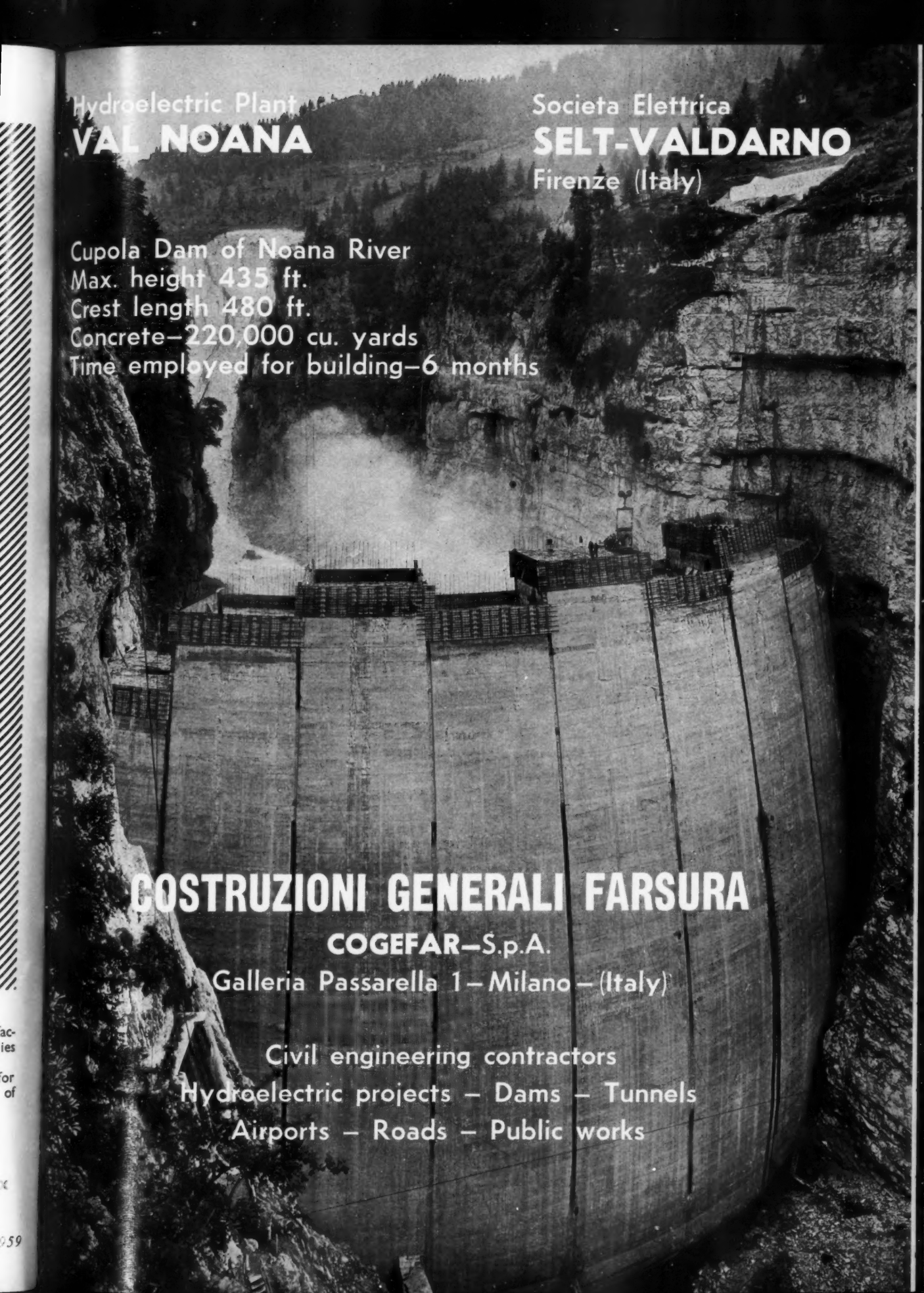
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Concrete–220,000 cu. yards
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for run-of-river plants, high-head developments or pumped-storage schemes—for vast or small power projects—Voith-Heidenheim is ready to put its technical competence and experience in hydraulic engineering at your disposal.

When you plan an installation, whether it is new or the rehabilitation or expansion of present facilities, we shall gladly, in response to your inquiry, prepare proposals for modern hydraulic equipment that will suit your individual requirements.

The photos show a few examples of more recent power stations where Voith equipment has been installed.

9904

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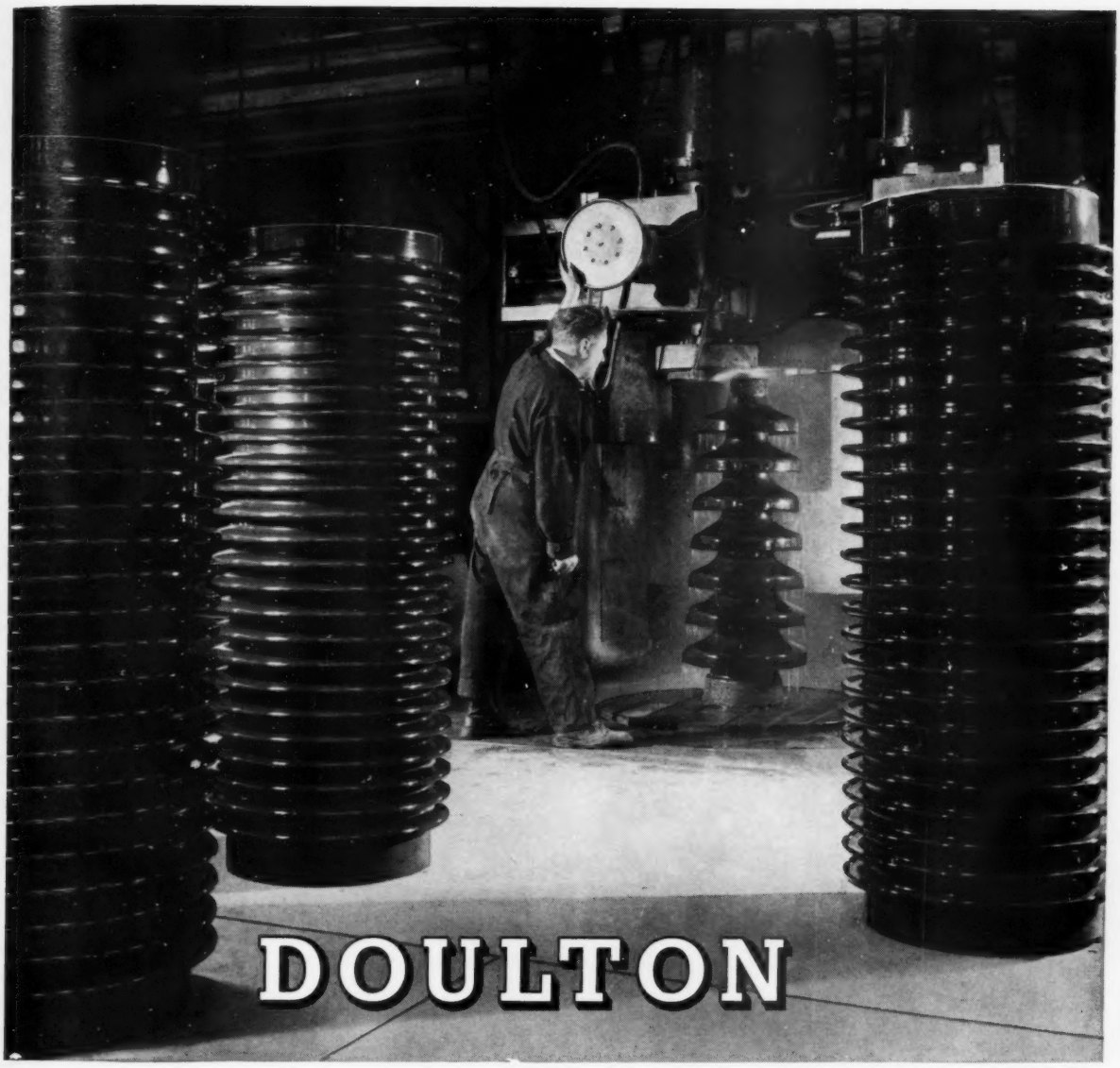

INSULATORS

A 132 kV Porcelain anti-fog cable ceiling end being ground to dimensional tolerances, as supplied to Pirelli General Cable Works Ltd. for use on C.E.G.B. contracts.

The three large 180 kV Porcelain bushings in the foreground are part of a consignment for the Canadian General Electric Co. Grinding and routine testing are the final stages before shipment to Toronto.

DOULTON INDUSTRIAL PORCELAINS LIMITED

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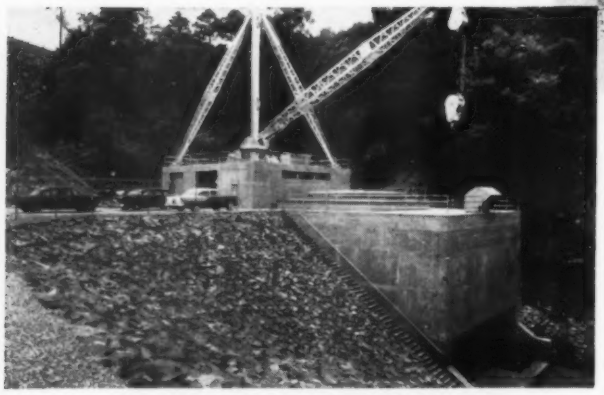
TVA relies on Leffel Turbines

AT WILBUR DAM

In 1950, when the Tennessee Valley Authority decided to expand the capacity of the Wilbur Dam Power station, a Leffel vertical shaft propeller-type turbine was selected. This turbine is rated at 11,500 HP, operating at 180 RPM under a net head of 67 feet.

AT CHATUGE DAM

The Leffel turbine at Chatuge Dam, another TVA installation near Murphy, North Carolina was also installed in 1955. This turbine is rated at 13,000 HP at 180 RPM under a net head of 100 feet.



AT NOTTELY DAM

At TVA's Nottely Dam on the Nottely River near Murphy, North Carolina the generator is driven by a Leffel turbine installed in 1955. This turbine is rated at 21,000 HP, operating at 180 RPM under a 124 foot net head.

These three turbines have given entirely satisfactory service from their initial installation.

. . . AND SO CAN YOU

Whatever your waterpower requirements you'll find Leffel turbines tops for efficiency and reliability. And Leffel engineers, drawing on nearly a century of experience, are available without obligation to help you plan the installation which will give you maximum power from available water.

Whether you're planning the rehabilitation or expansion of existing facilities or the construction of a completely new installation, be sure to find out what Leffel can do to help you.

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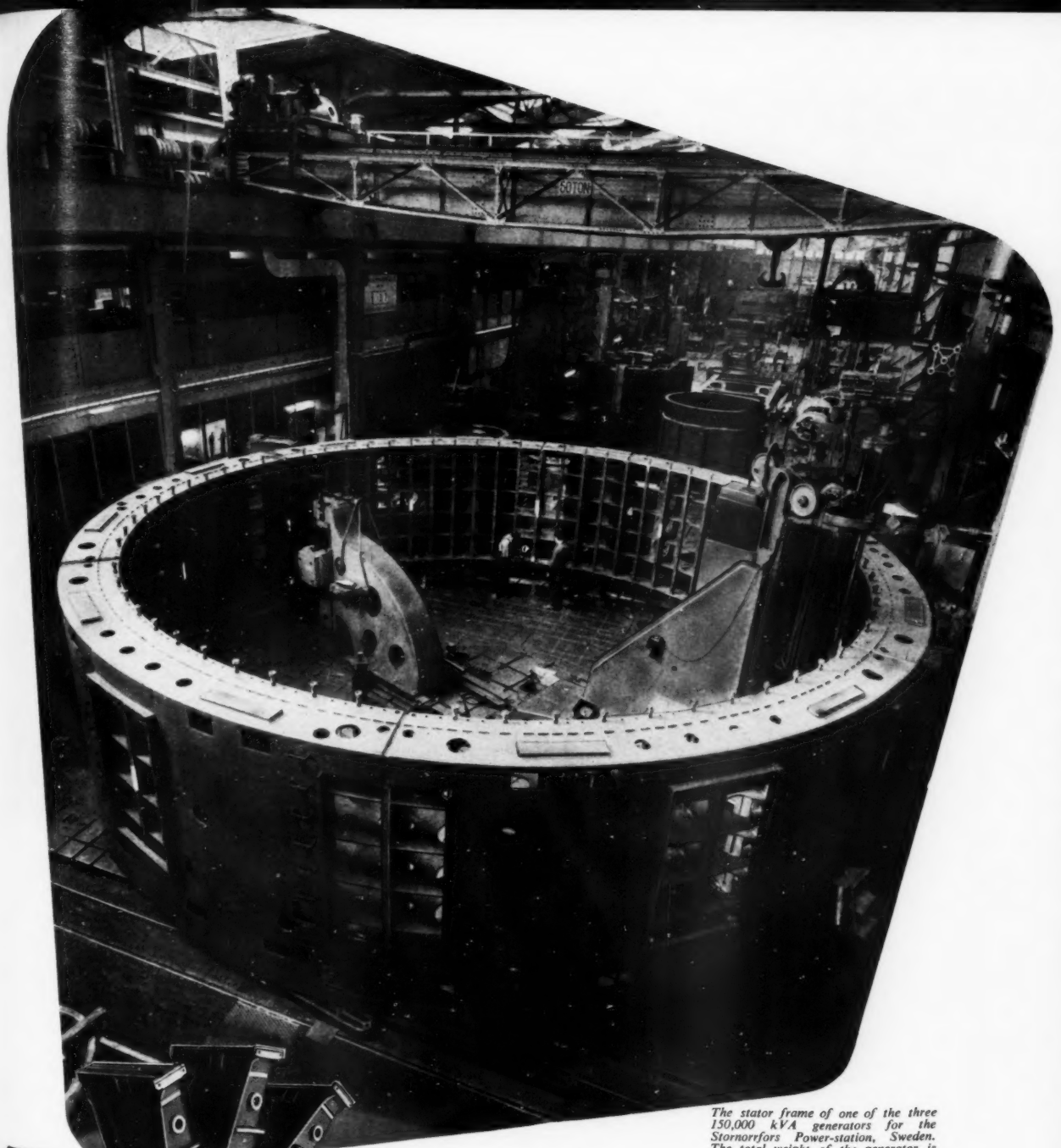


The stator frame of one of the three 150,000 kVA generators for the Stornorrfors Power-station, Sweden. The total weight of the generator is 925 tons.

The rotor spider with a diameter of 8.36 metres and a weight of 86 tons for one of the Stornorrfors generators.

Power in the making

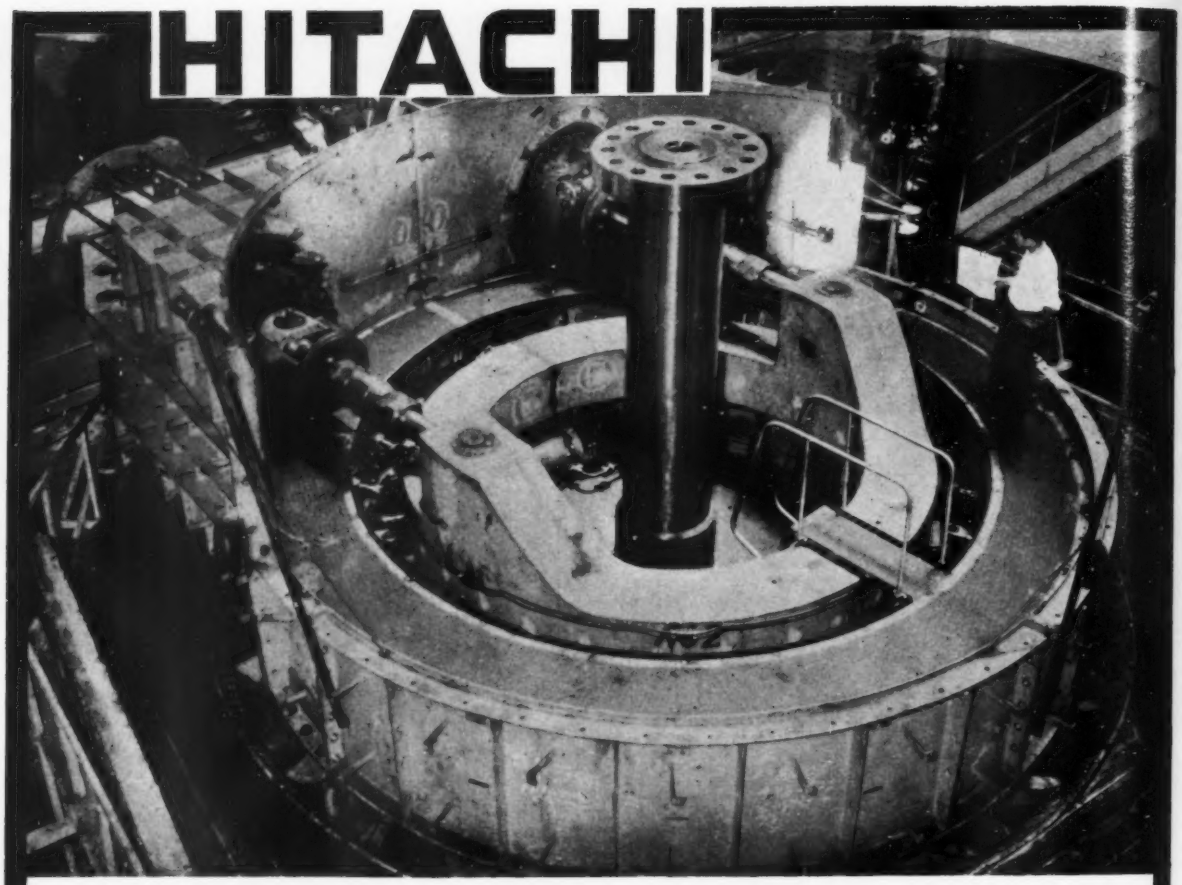
In Sweden water power is abundant, and practically all of the country's power production—which is notably high—emanates from ASEA Generators.

The building of electric equipment for hydro-electric power stations has been one of ASEA's main activities for more than 75 years. Recently ASEA has supplied three 150,000 kVA generators for a Swedish Power Station, and for stations abroad ASEA-units of water wheel generators built after the war, or under way, amount to a total of about 2,000,000 kVA.

A visit to these stations would mean a full round-the-globe trip!—So we think ASEA might be able to solve even your power problems. We have the experience and the resources.

Västerås **ASEA** Sweden

HITACHI



Main specifications:

Output: 29,840 kW (40,000 HP), Discharge: 115·55 m³/s (4,080 ft³/s) maximum
Effective head: 29·87 m (98 ft), Speed: 167 r.p.m.

Two more turbines for India!

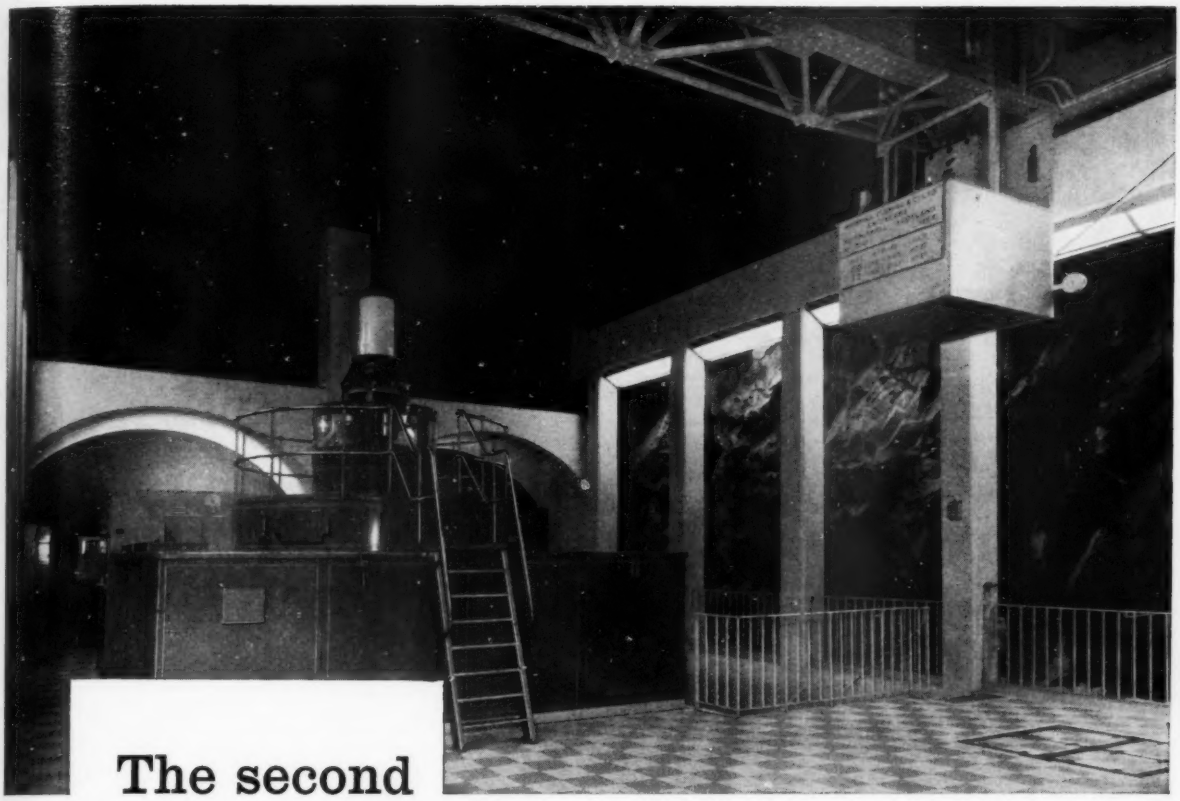
Yet another proud Hitachi achievement has been the completion of two 29,840 kW (40,000 HP) Fixed Blade Propeller Turbines for the Ganguwal and Kotla Power Stations near India's massive Bhakra Dam in the State of Punjab.

Both Ganguwal and Kotla have been in operation since 1951 and these two Hitachi turbines represent additional output to that provided by existing equipment. Many technical problems were posed by this and the fact that the draft tubes were already installed, but a series of model tests have demonstrated their high efficiency.

Another of the interesting features that distinguish these turbines was that welding construction was applied to all parts except the runner which, out of consideration for transportation problems was made in sections for assembly at site.

Hitachi, Ltd.
Tokyo Japan

Cable Address: "HITACHY" TOKYO



The second underground power station

The St. Fillans Power Station is the second major power station constructed completely underground in the British Isles. Both this and the first major underground station were included in contracts carried out by the Mitchell Construction Co. Ltd.
St. Fillans Power Station forms part of the St. Fillans section of the North of Scotland Hydro-Electric Board's Breadalbane scheme. Mitchell Construction were responsible for the construction on this scheme of two generating stations, 19 miles of tunnelling and numerous intake works and pipe aqueducts.

Consulting Engineers: Sir M. MacDonald & Partners

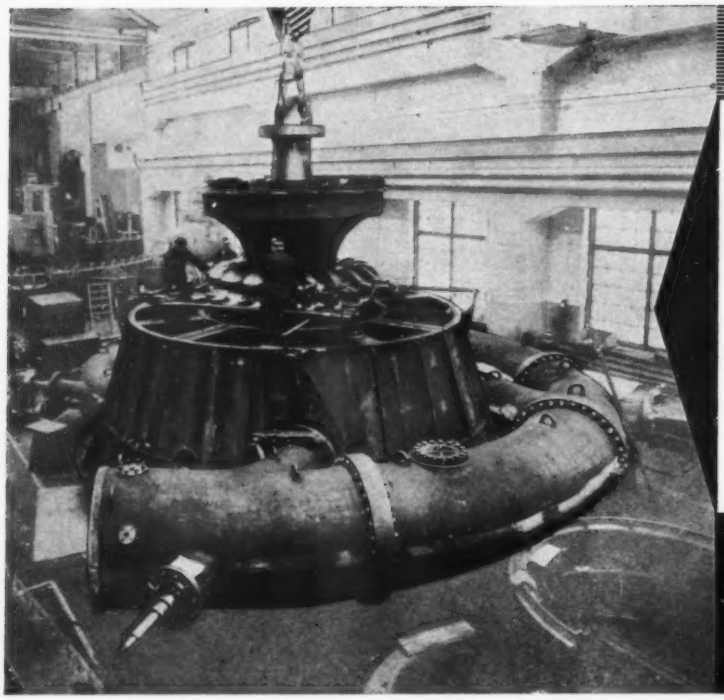
MITCHELL CONSTRUCTION

Building and Civil Engineering Contractors

THE MITCHELL CONSTRUCTION COMPANY LTD

Wharf Works, Peterborough

P.3049

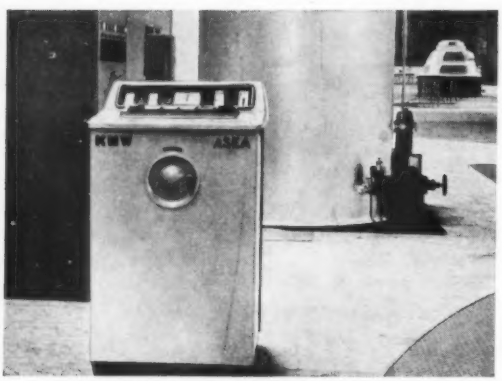


The Largest Vertical

PELTON Turbines

in Norway are KMW Turbines

In the last half of 1958 the first of the two Pelton Turbines ordered from KMW by the Hjartdöla Power Station in Norway was put into operation; the second unit was put into service in May, 1959. Each turbine has a nominal output of 71,000 H.P. at 555 m (1,820 ft.) net head and 428 r.p.m.



Both the turbines are equipped with the ASEA-KMW system of electro-hydraulic governing. Impulses from the electronic regulator are converted into mechanical motion in the electro-hydraulic actuator. This motion is transmitted via a separate servomotor to the deflectors and the control valves of the needle servomotors, which are linked by a conventional system of cams to provide the correct combination of controls.

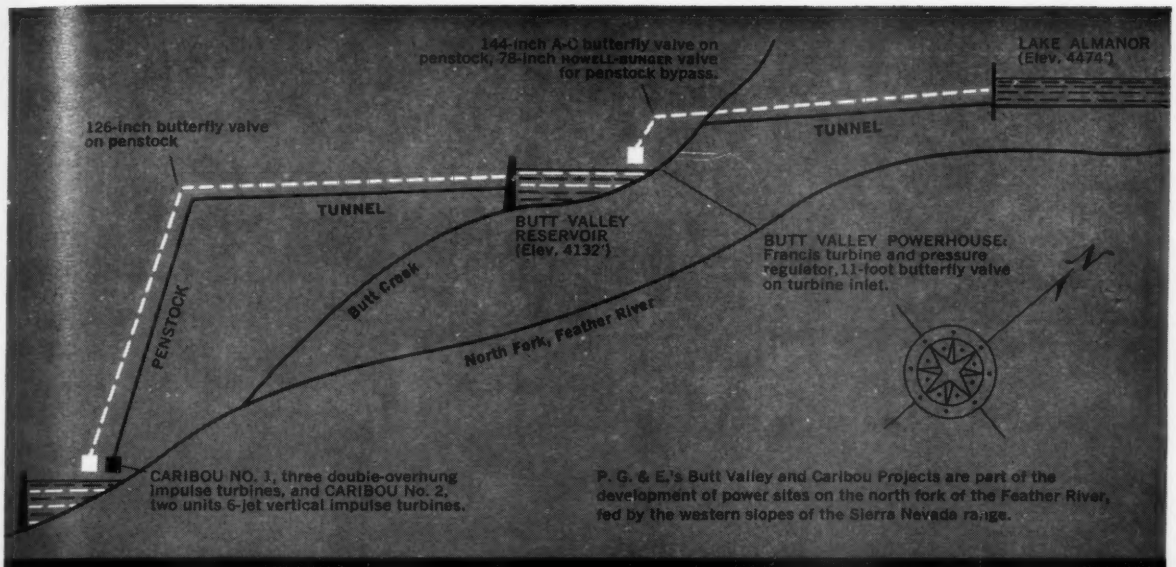
This is the first time that ASEA-KMW electro-hydraulic actuators have been used for Pelton turbines, and this regulating system has proved to operate perfectly. Alterations in speed when throwing-off load are very small and the return to normal revolutions is remarkably quick.

AB KARLSTADS MEKANISKA WERKSTAD

Karlstad - Sweden

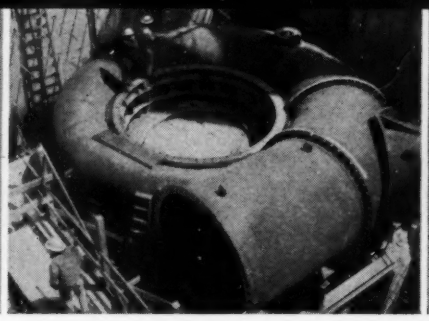
Representatives in Canada: A. Johnson & Co. (Canada) Ltd.
607 Shell Tower Building, Montreal 2.

KMW

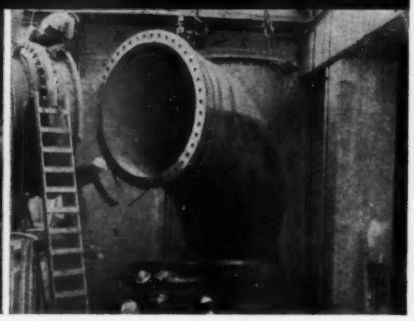

144-inch A-C butterfly valve on penstock, 78-inch HOWELL-BUNGER valve for penstock bypass.
LAKE ALMANOR (Elev. 4474')
126-inch butterfly valve on penstock
TUNNEL
TUNNEL
BUTT VALLEY RESERVOIR (Elev. 4132')
BUTT VALLEY POWERHOUSE: Francis turbine and pressure regulator, 11-foot butterfly valve on turbine inlet.
PENSTOCK
Butt Creek
North Fork, Feather River
CARIBOU NO. 1, three double-overhung impulse turbines, and CARIBOU No. 2, two units 6-jet vertical impulse turbines.
P. G. & E.'s Butt Valley and Caribou Projects are part of the development of power sites on the north fork of the Feather River, fed by the western slopes of the Sierra Nevada range.

The 11-foot butterfly valve on the Butt Valley turbine inlet is shown being lowered into position.

Turbine spiral case in position at Butt Valley prior to the installation of the turbine runner.



Pressure regulator inlet elbow being positioned. Spiral case is at the left and energy absorber and discharge chamber below.



Allis-Chalmers Turbines, Pressure Regulator and Valves Installed at Butt Valley and Caribou Powerhouses, No. 1 and No. 2.

At Pacific Gas & Electric Company's new Butt Valley Powerhouse development, an Allis-Chalmers vertical Francis turbine with pressure regulator, two butterfly valves and one HOWELL-BUNGER valve are installed. This turbine is rated at 55,000 HP at 200 rpm under a 280-foot rated head.

A stalling-type pressure regulator, with provision for quick and simple conversion to synchronous bypass control, it's attached to the spiral case. Designed for a minimum opening time of 3 seconds, the pressure regulator is required because of the 5703-foot long penstock and the relatively remote location of the surge tank. An 11-foot A-C butterfly valve is located at the turbine inlet, while the other butterfly valve will be used as a penstock shutoff valve. The HOWELL-BUNGER valve serves as a penstock bypass.

Down stream from Butt Valley is Caribou No. 1 Powerhouse, with 3 Allis-Chalmers horizontal, double-overhung impulse units, 30,000 HP each, two units paralleled 1921, the third unit paralleled 1924, and Caribou No. 2 Powerhouse, completed along with Butt Valley in 1958. Caribou No. 2 has two Allis-Chalmers vertical, 6-jet impulse turbines, each rated 76,000 HP at 1110-foot head at 240 rpm. At each turbine inlet is a 54" rotary valve of latest Allis-Chalmers manufacture.

You can look to Allis-Chalmers for the latest in design, engineering and manufacturing of a complete line of hydraulic turbines and their accessories. For further information, contact your local Allis-Chalmers office or write Allis-Chalmers, Hydraulic Division, York, Pa.

HOWELL-BUNGER and ROTOVALVE are Allis-Chalmers trademarks

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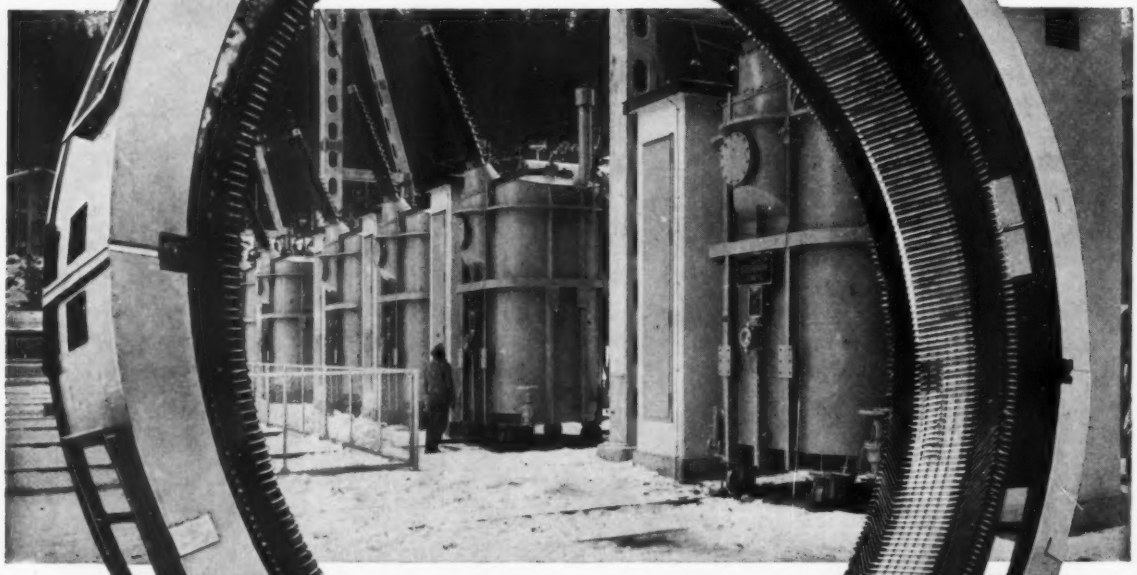
Hydro-Electric Power Station Equipment



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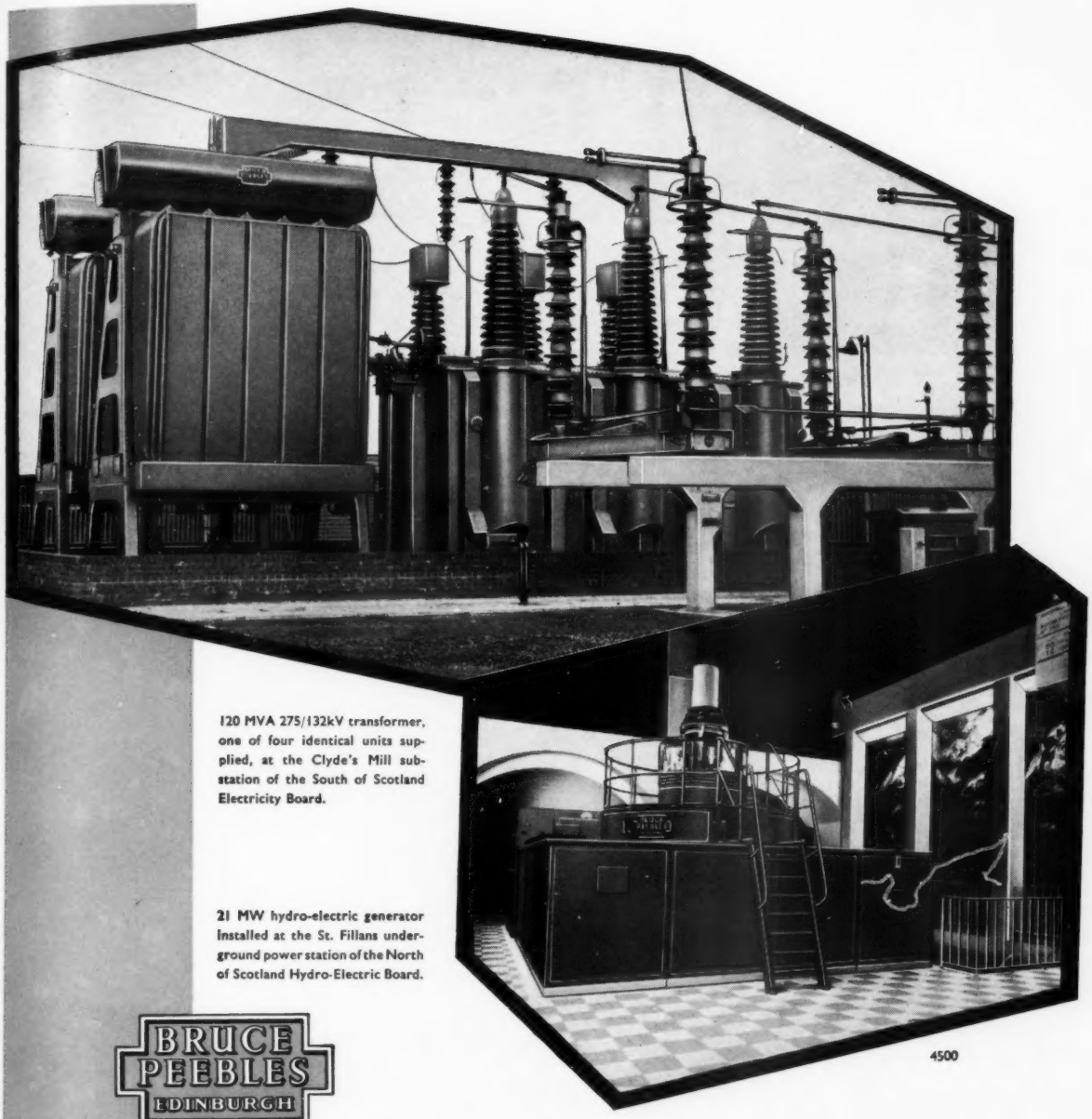
S. A. des Ateliers de Sécheron, Geneva



120 MVA 275/132kV transformer, one of four identical units supplied, at the Clyde's Mill substation of the South of Scotland Electricity Board.

21 MW hydro-electric generator installed at the St. Fillans underground power station of the North of Scotland Hydro-Electric Board.

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Bruce Peebles & Co. Limited, are playing a leading role in the manufacture and supply of large hydro-electric generators and transformers up to the largest sizes and highest voltages, for important power development schemes at home and abroad.

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facing the runner hub with stainless steel "Bricks"

This "upstream-bulb" axial unit can work as a turbine or pump, irrespective of flow direction

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head : 5, 50 m
speed : 88, 25 r. p. m.
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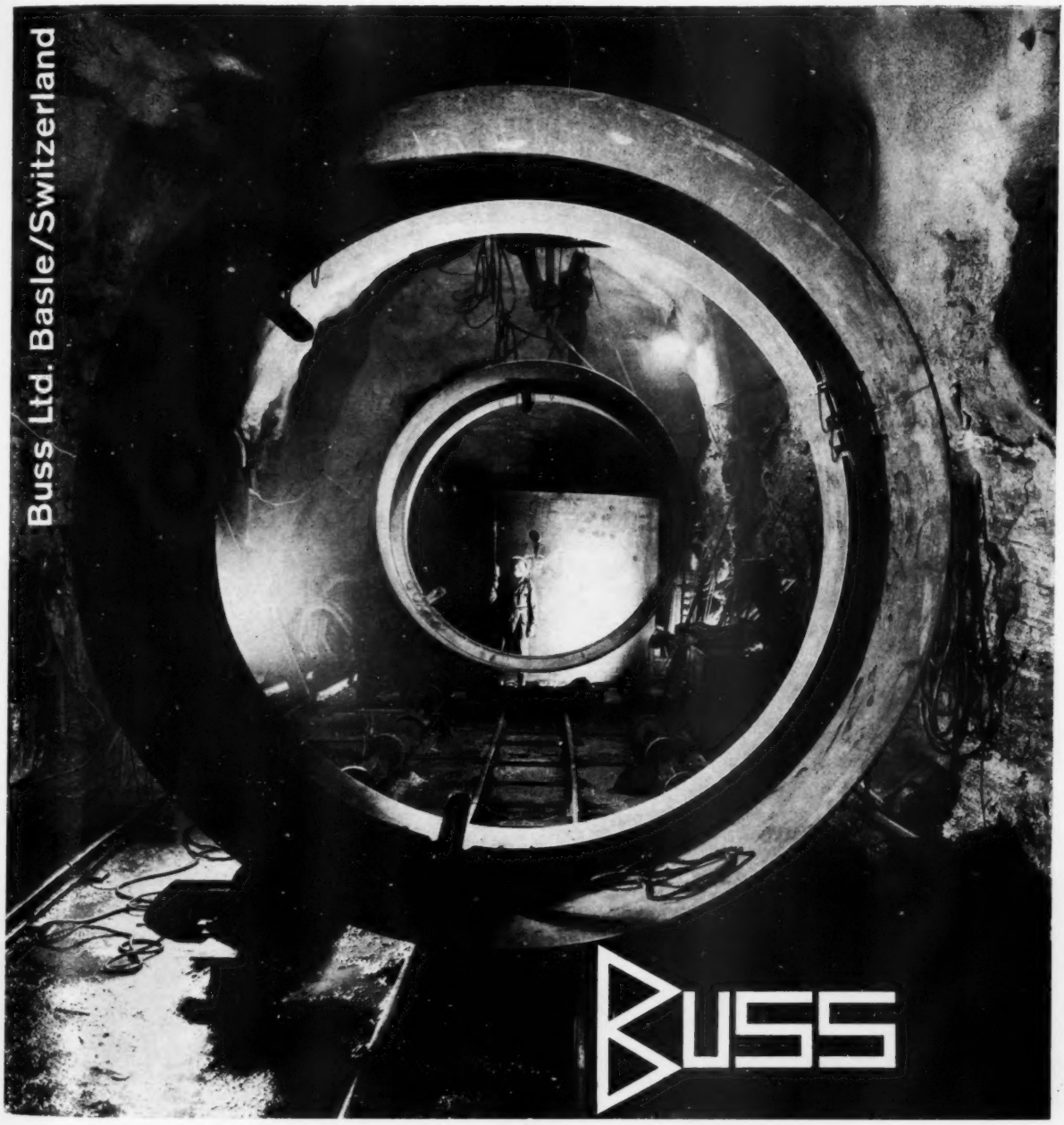

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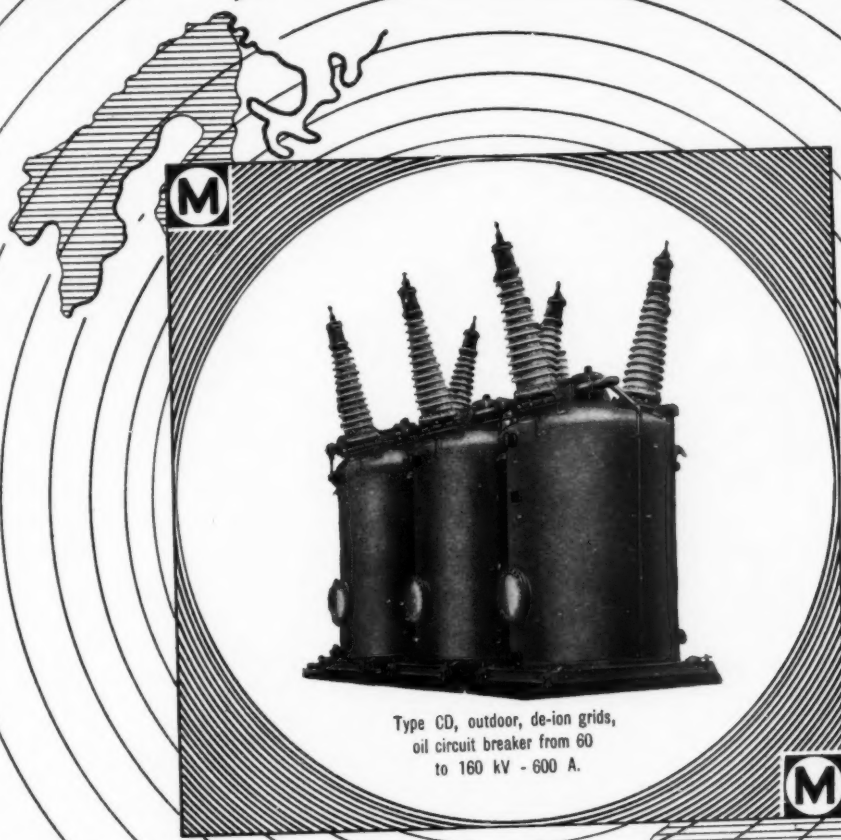

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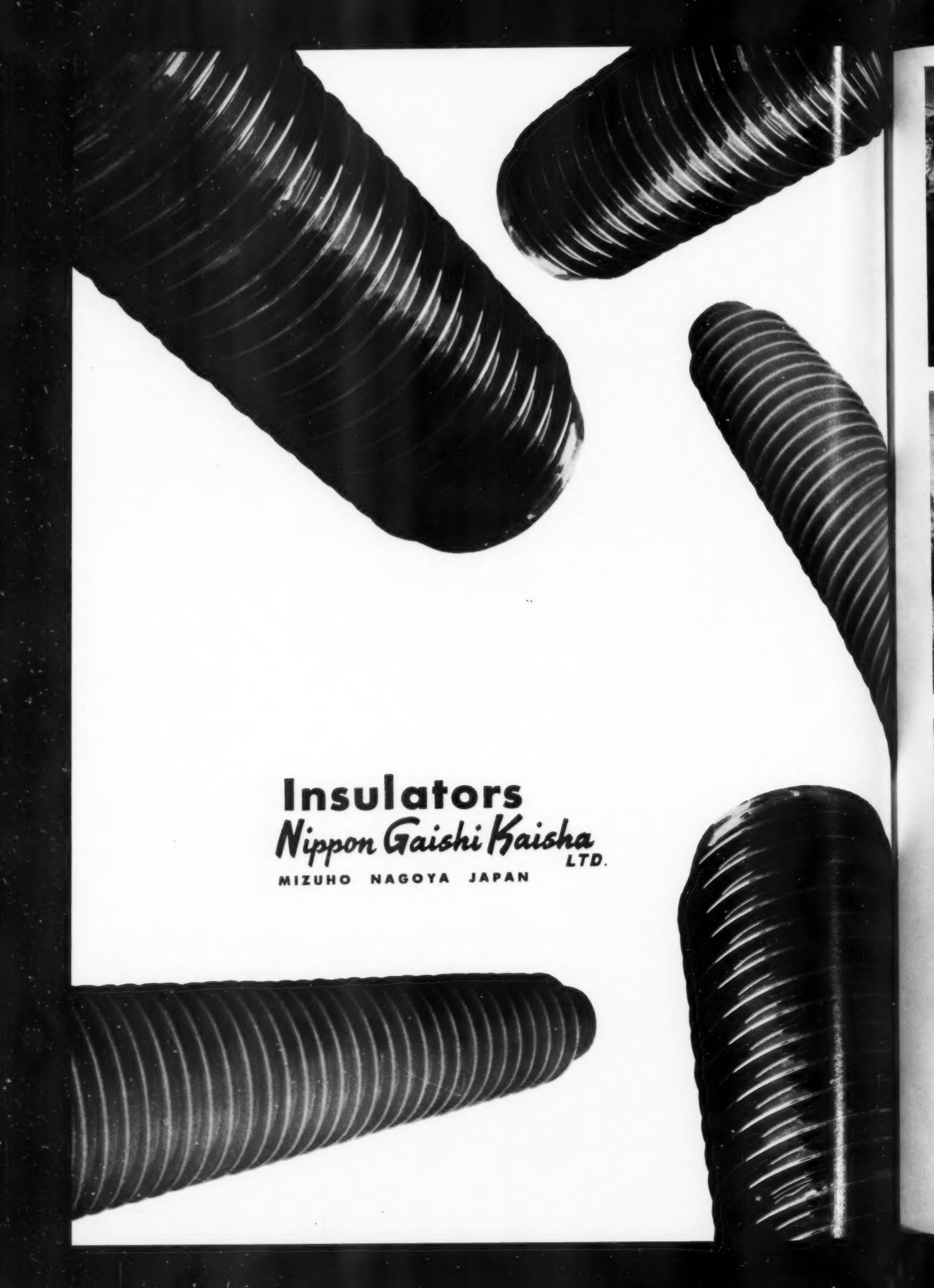
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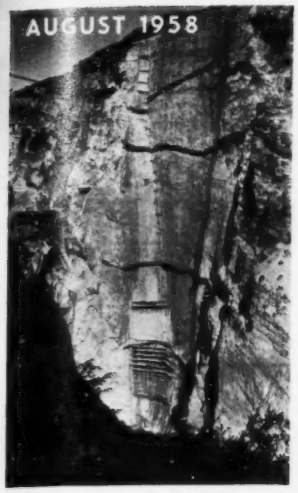

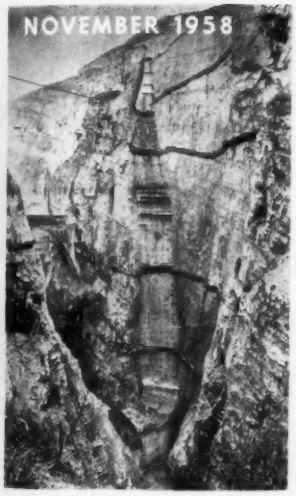

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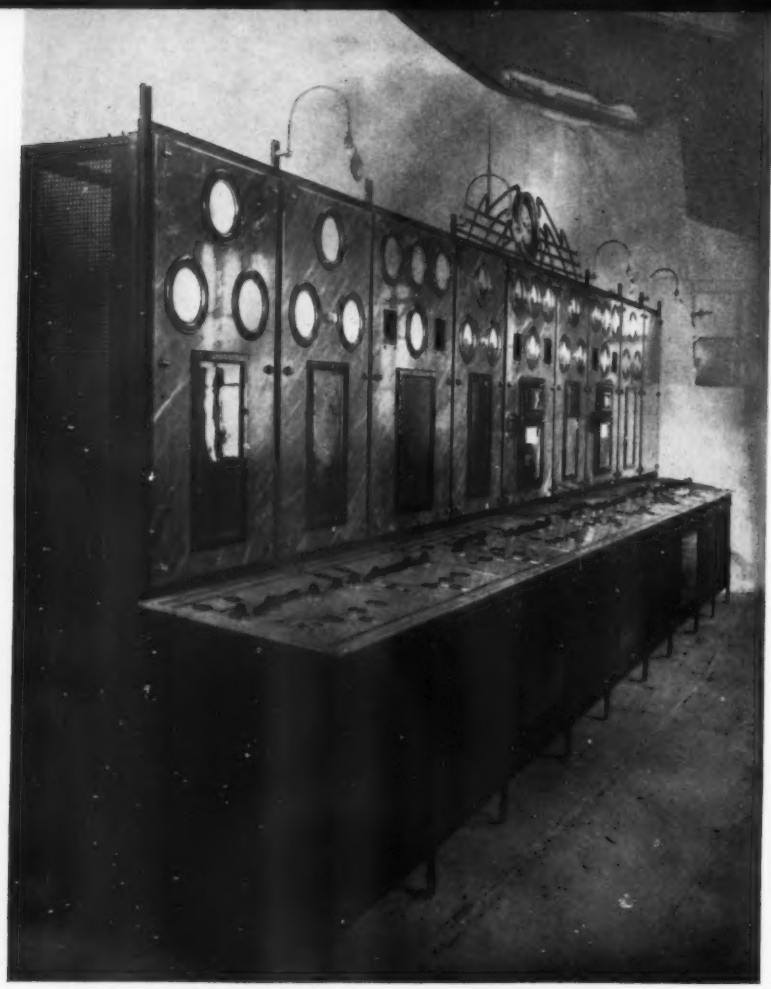

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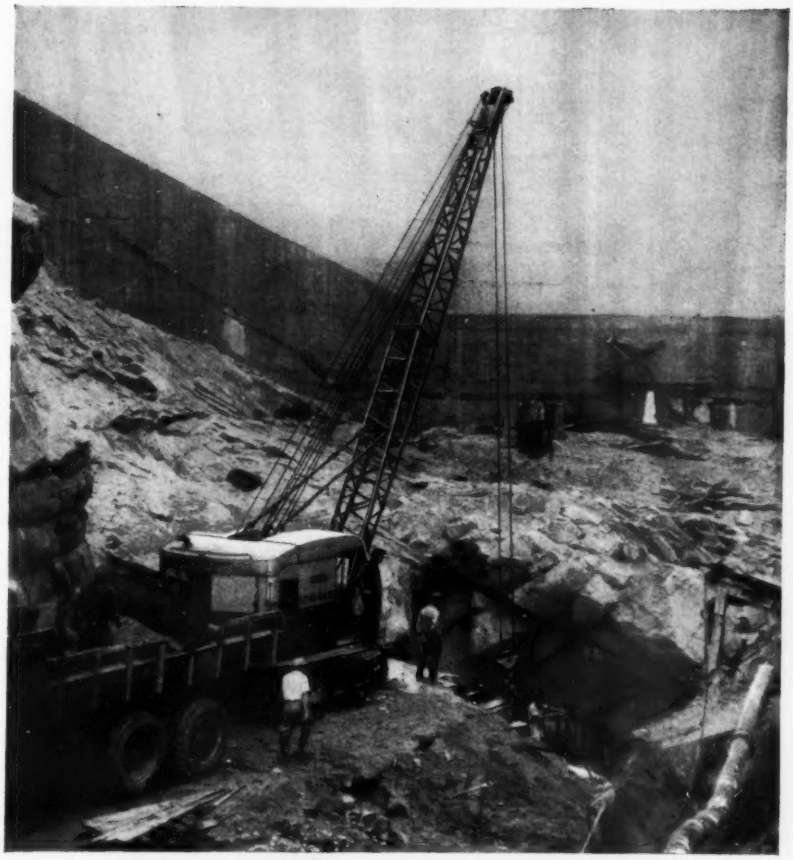

19-RB operating as a skip crane at the foot of the Chicamba Dam in Portuguese East Africa. This concrete dam spans the River Revue near the Rhodesian border. The rock is broken up by compressed air drills and hand loaded by native labour into Euclid 9 yd. trucks.

22-RB lifting crane on the extension to a Hydro-electric scheme at Dalwhinnie, Scotland.

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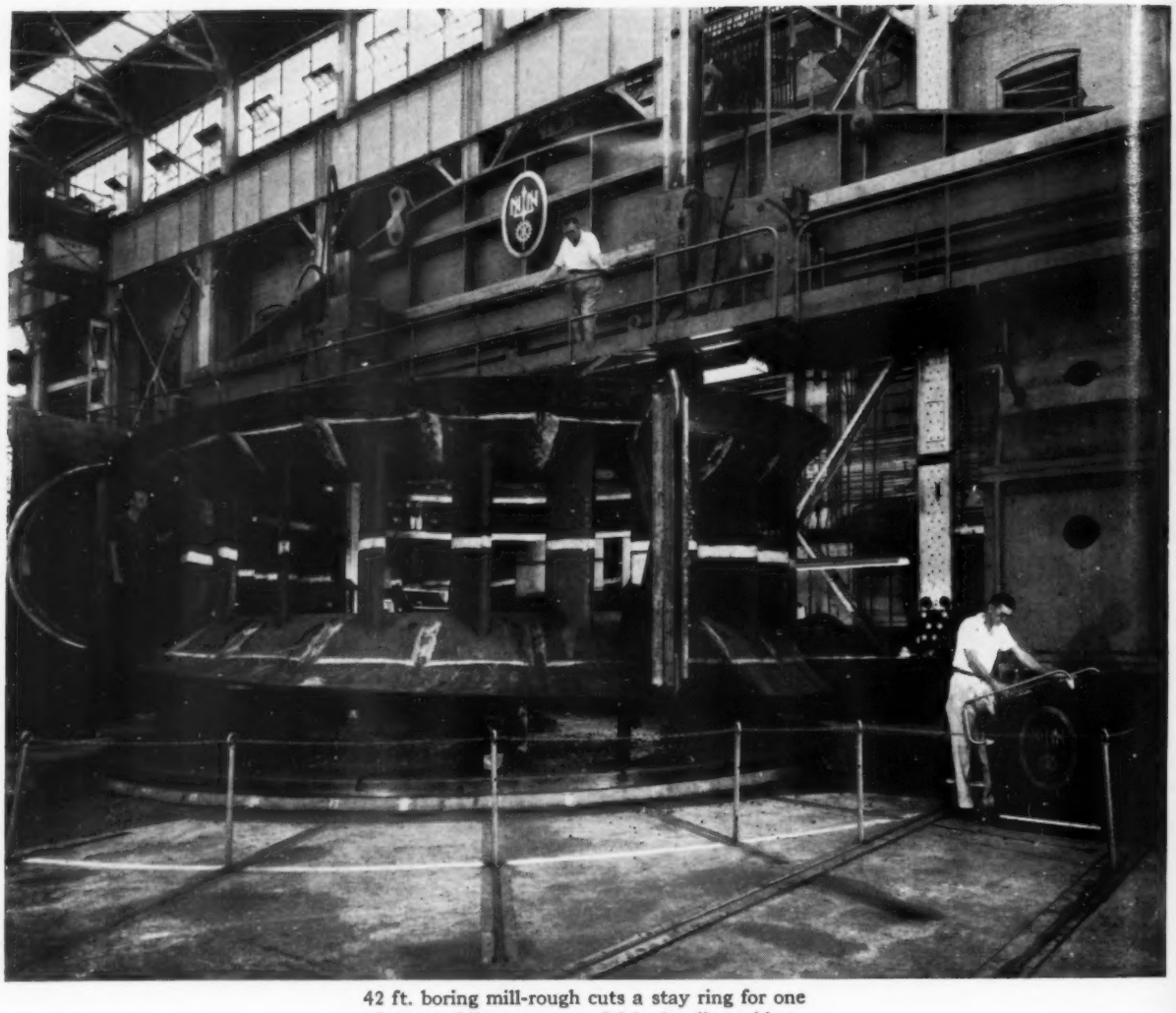

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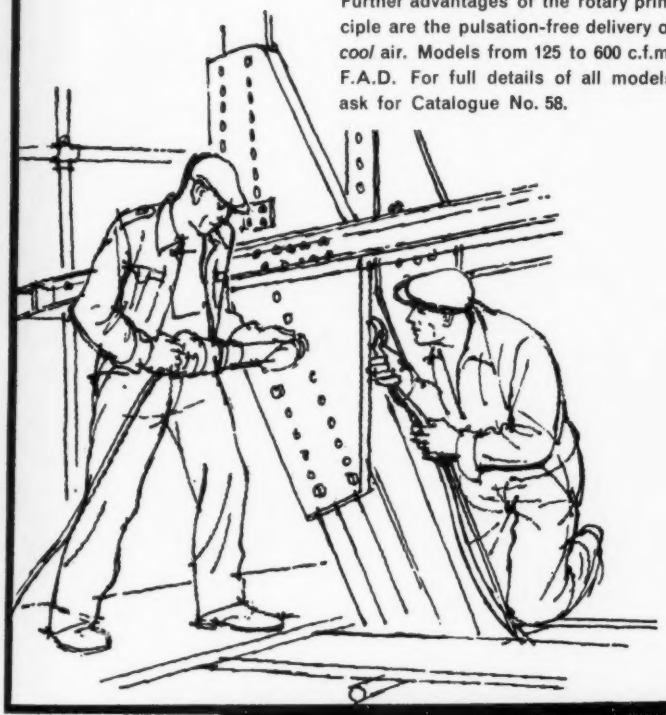
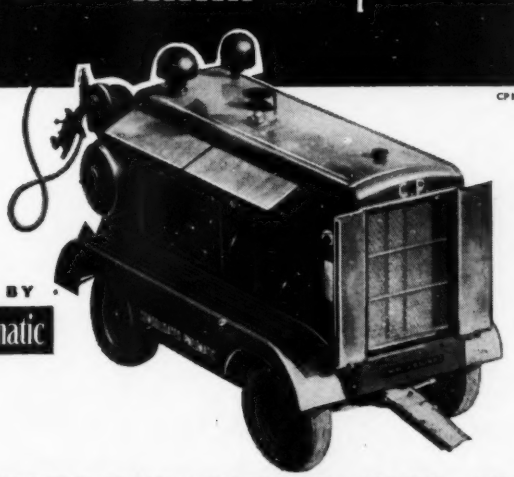


POWER VANE
ROTARY COMPRESSORS

Cost less to operate

CP186



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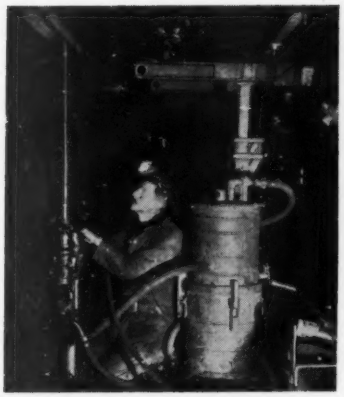

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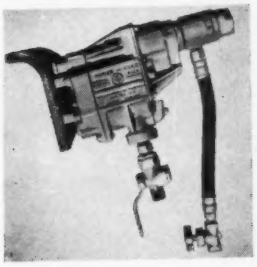

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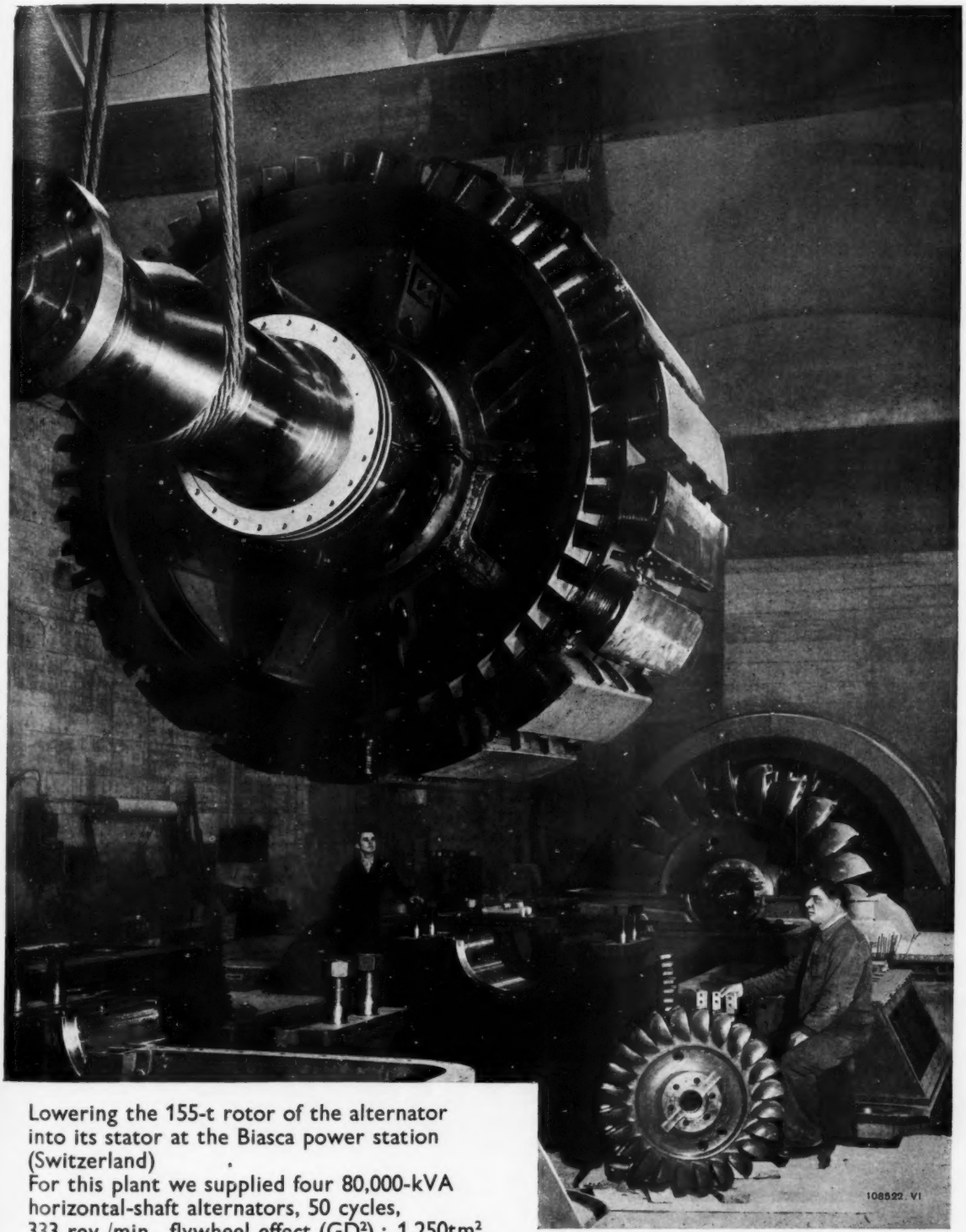

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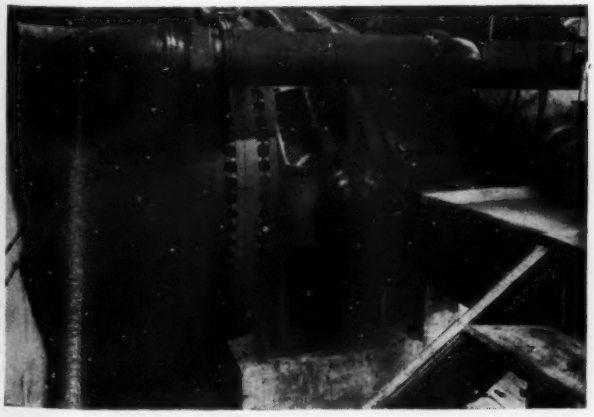

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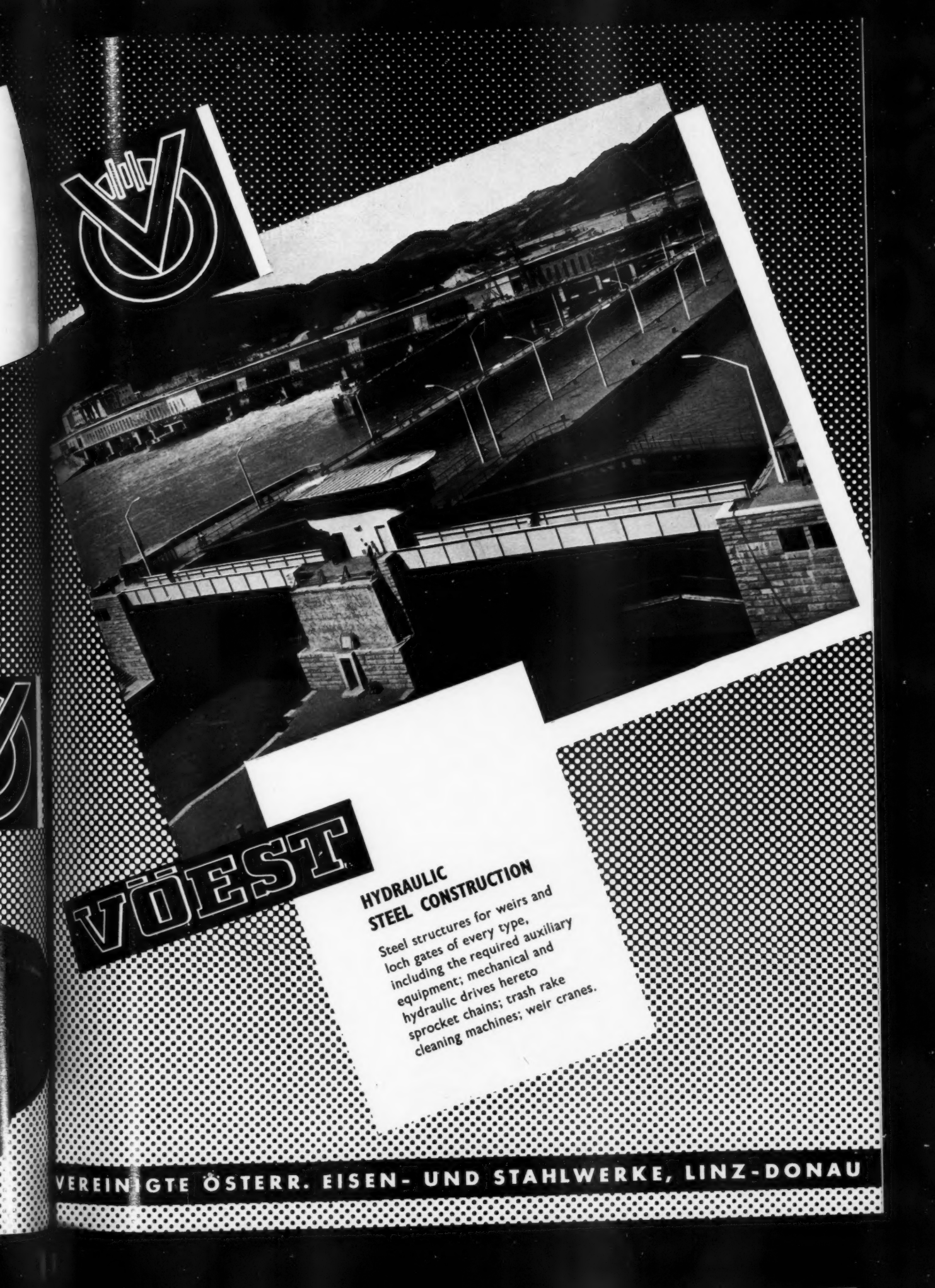
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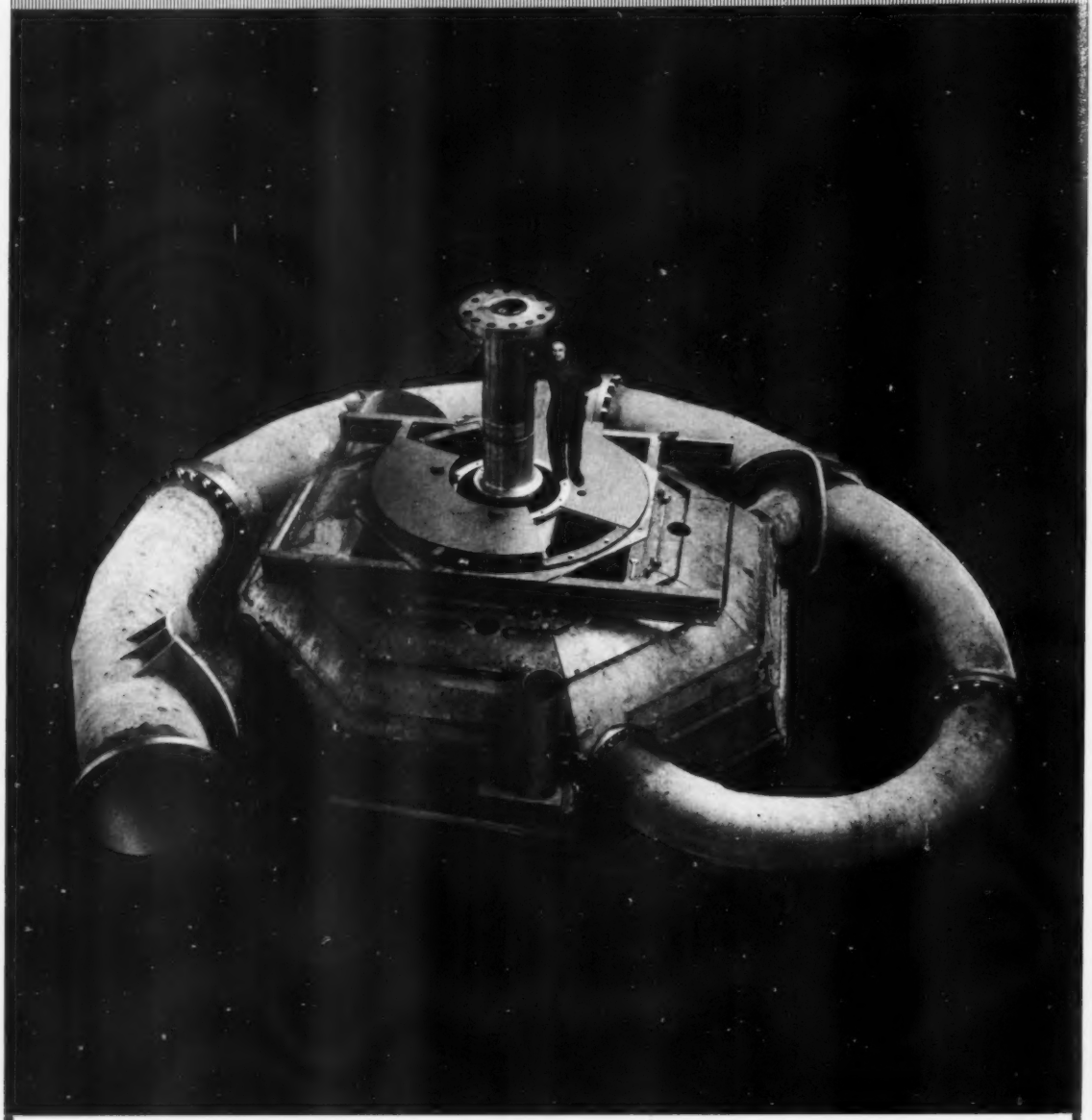

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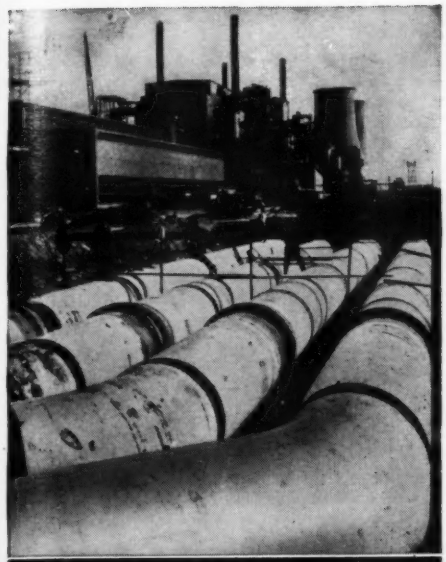

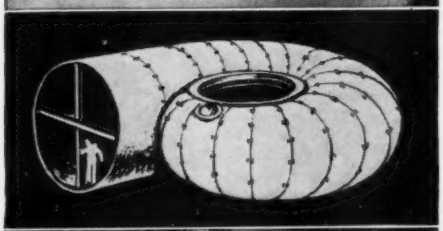

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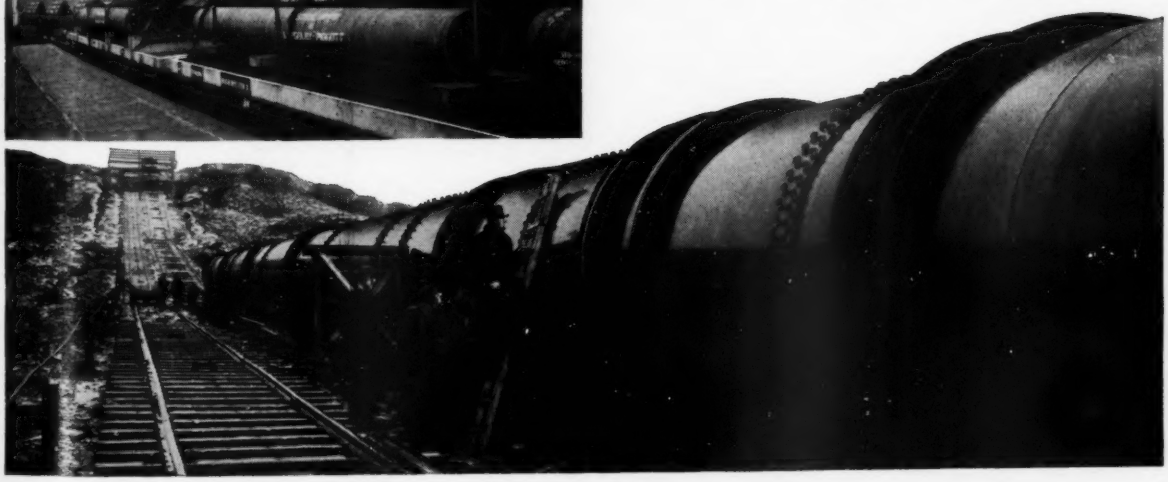

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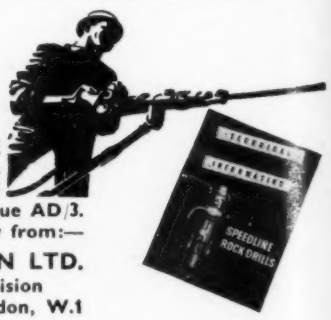

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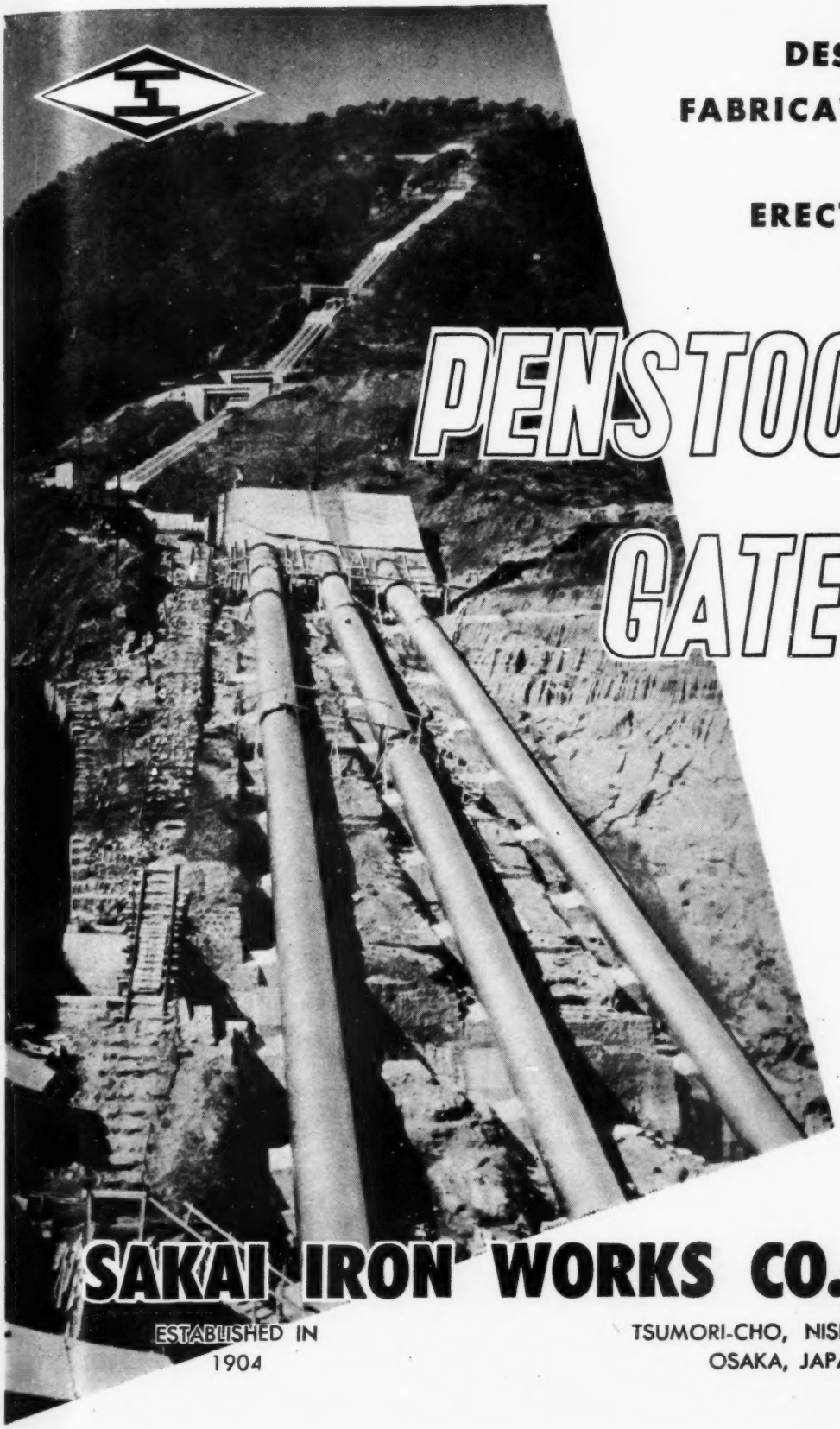

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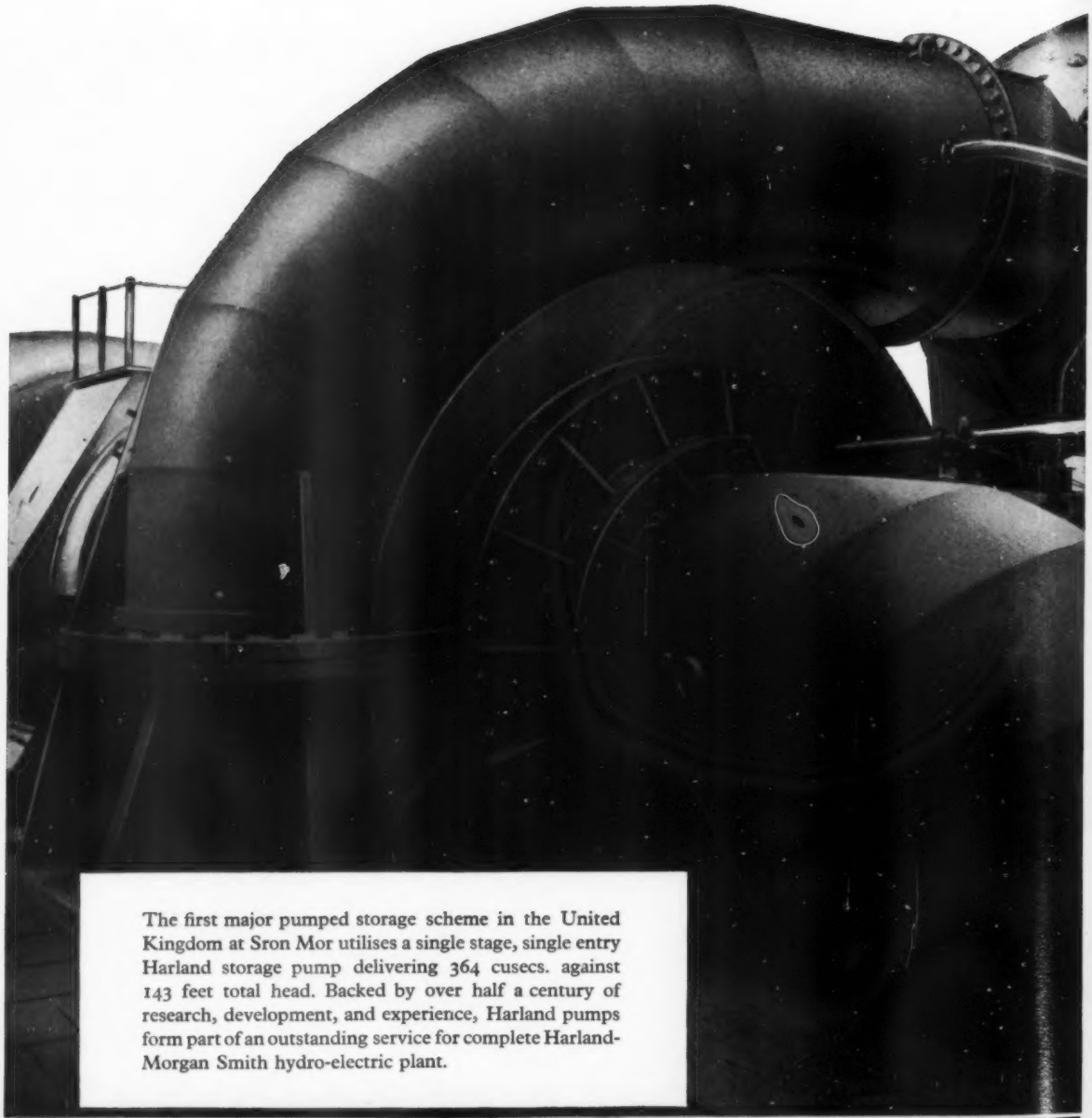

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New power for a Dominion from the Roxburgh Hydro-Electric Project

New Zealand's largest hydro-electrical undertaking, the Roxburgh project, will eventually feed an additional 320,000 kilowatts of electricity into the South Island grid system. The 1,200 ft. Dam, which is the largest ever built in the Dominion, was completed far ahead of schedule by Cubitts' New Zealand Co., in association with Conrad Zschokke Ltd., Geneva, and Downer & Co. Ltd., New Zealand.

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

The Problem of Load Factor

IN planning a country's electricity supply, once sufficient generating plant has been provided, the next most pressing problem is that of improving the load factor. In practically all parts of the world the demand for electrical energy increases at the rate of 7% per annum, which obviously involves doubling the plant capacity every ten years. The share of national investment which has to be devoted to increasing the generating capacity is becoming so great as to focus attention on the necessity to keep this expensive plant at work for more than the usual average of 40% to 45% of its available time in any month or year.

It is, of course, a problem which presses on the operators of thermal plants much more hardly than is the case with hydro-power installations. For example, boilers kept hot and turbines kept warm and slowly barring round to await the next load period consume a considerable amount of fuel. But even if no water is spilled when the gates close to shut down a single water turbine or even the whole of a hydro station, as the load drops away, the equipment is still not earning the return on the invested capital which it should and could earn if the load factor could be improved.

In countries such as Great Britain, where a programme of nuclear power-plant construction is under way, it has been unequivocally stated that the price of energy from nuclear power can only compete with that from "conventional" sources if the load factor on the expensive atomic stations is kept as high as possible; a figure of 80% has often been mentioned. This will inevitably mean that the load given to the remaining stations is decreased, thus reducing their profitability.

In considering the possibilities of improving the load factor there are two aspects to be taken into account—external and internal. From the external point of view, the load curve is fixed by the demands of the consumer. Can he be asked to change his demand to suit the economics of the power supply industry? Looked at from the standpoint of possible changes in closely integrated and long-established power systems, like those of the U.S.A. and many European countries, it seems that very little can be done to improve load factor by increased inducements in the form of cheaper off-peak tariffs. Industry is not inclined, in general, to risk unfavourable labour reactions by endeavouring to arrange shift working; and it is only in a very few industries, such as those concerned with large-scale metallurgical and chemical processes, that a relatively small reduction in electric power rates would in any case become an attractive proposition to the management. There is no known means of persuading the domestic consumer to change his habits of usage for the sake of the electricity-supply organisation, although intensive sales campaigns can make him purchase types of equipment, such as thermostatically controlled heaters, which ultimately bring a slightly better shape to the load curve.

From the internal viewpoint, can the power-supply industry pull up its load factor by tugging at its own boot straps? The hydro-electric engineer offers the first answer: the use of pumped storage. This aspect of water-power engineering is receiving increased attention in many parts of the world, but it is obvious that there are topographical difficulties which in many cases are impossible to solve. Nevertheless, it may well be the case that sites which are not considered as economic for normal hydro-power generation could be reconsidered, with the additional economic spur of load-factor improvement, if looked at mainly from the pumped-storage point of view. Even pumping sea water to an artificial inland reservoir need not be so impracticable as it has seemed in the past, and in fact installations of this type have actually been planned for the island of Majorca, and for South Australia. But although pumped storage can have an important influence on the load curve, it is not capable of universal application.

Another approach to load-factor improvement lies in the hands of those responsible for long-term planning of power development in the less-industrialised countries. Long-range interconnection between groups

of consumers with differing clock times and differing patterns of industrial and domestic energy utilisation is the surest possible way of improving load factor. This applies with particular emphasis to international connections. With the electrical map of Europe showing power lines weaving their routes across frontiers at every turn, international electrical interconnection can be seen to be a workable proposition, particularly when its aim is to improve operational characteristics on the interconnected systems, and not to make any one of them solely dependent on the others. But whether national boundaries are involved in an interconnection scheme or not, the essential point never to be overlooked is that future energy planning, now carried out with very great care, still needs an expansion of its horizons if the enormous amounts of capital involved are to be utilised properly. In the future, very few power schemes of any size should be considered alone. Only by a continuous widening of the horizons of power development, and by overcoming artificial barriers between nations, so as to link together the greatest possible number of diverse energy-usage patterns, will it be possible in the future for the power engineer to justify his huge requisitions on the treasuries of the world.

Stornorrfors Officially Opened

STORNORRFORS station on the Ume river was officially opened on September 13 by Princess Sibylla, of the Swedish Royal House, in the presence of about 400 guests. Stornorrfors, at a present capacity of 375 MW, is already the largest station in Sweden, but space is available for a fourth set which will bring the capacity to 500 MW. The individual sets, which are rated at 150 MVA, are some of the largest Francis machines in the world, and the tailrace tunnel, which has a cross section of 400 sq.m., is the largest tunnel in the world by a wide margin.

Stornorrfors forms an integral part of the intensive development of the Ume river by the Swedish State Power Board in conjunction with certain private interests. By 1965 no less than 13 stations will be in operation below Lake Storuman and will supply 16% of Sweden's electrical demand. It is hoped to publish in this journal in due course an account of the Ume river development, together with a detailed description of Stornorrfors station.

Contracts for Scottish Hydro-Electric Scheme

THE North of Scotland Hydro-Electric Board have placed the main civil engineering contracts for the construction of their Strathfarrar and Kilmorack Scheme (Constructional Scheme No. 30) amounting to the best part of the allotted cost of £14 million. The scheme will produce annually 268 million kWh from a catchment area of 350 sq. miles in the Counties of Inverness and Ross & Cromarty. The main dam will be at Loch Monar in Strathfarrar with a secondary dam to its west. Water from Loch Monar will pass through a tunnel 5¾ miles long to a power station near Deanie at the west end of Loch Beannachran. Below the outlet of Loch Beannachran there will be another dam from which water will pass through a 3½-miles-long tunnel to a power station on the River Farrar at the Culligran Falls. There will also be two other dams and two power stations on the River Beauly. The upper dam and power station will be near the Crask of Aigas and the lower dam and power station at Kilmorack.

The Mitchell Construction Company Ltd., have been awarded the contracts for the construction of the Monar dams, the Deanie tunnel and the Culligran tunnel and power station. Duncan Logan (Contractors) Ltd., Dingwall, have been awarded the contracts for the construction of the Beannachran dam and the Misgeach aqueducts. A. A. Stuart & Sons (Glasgow) Ltd., will construct the dams at Aigas and Kilmorack and the power stations which are integral parts of these structures. Riley & Neate Ltd. have been awarded a contract for the erection of the 132,000 volt transmission line which will carry the output of the power stations to the Highland Grid near Beauly. This line will, at the outset, provide power for the constructional works.

The civil engineering consultants are Sir William Halcrow & Partners; the electrical and mechanical consultants are Messrs. Kennedy & Donkin.

Water Research Foundation of Australia

THIS Foundation was conceived in November, 1955, after a symposium on "The Water Resources of Australia—Their Control and Development" which was held at the University of New South Wales. The basic aims were to investigate the water resources of Australia, to encourage and promote research into the development and uses of these resources and to carry out whatever is necessary for their control. The intensive efforts put in by its members saw the Foundation firmly established and well on the way to achieving its aims in such a short period of time. Financially the Foundation depends mainly on donations and so far a sum of £30,000 has been raised which enabled several grants to be made to the Universities of Sydney, Melbourne and New South Wales to conduct researches into a diversity of water problems.

Among the many important projects being conducted one stands as being of interest to the readers of WATER POWER. This project is on the "Stability of Self Spillway Rockfill Dams" being carried out by Melbourne University which involves the devising of a stable embankment mainly of rock to act as the wall of a dam. The idea is that it will remain stable during flooding whence the surplus flood-water can pass over the dam as it does in some concrete dams.

International Bank Report

THE World Bank has announced that its loans in the past fiscal year amounted to $703 million and that its reserves reached a record figure of $420 million. This was the second year in succession that the Bank's lending exceeded $700 million, and confirmed the Bank's belief of a year ago that its operations had reached a new plateau. The fiscal year also registered new records in the Bank's disbursements and sales of loans. Net income during the year rose by 10% to a new high level of $46·5 million.

These facts are recorded in the Bank's annual report, which was presented recently by President Eugene R. Black to the Annual Meeting of the Bank's

Board of Governors, in Washington, D.C. The report describes the action being taken by member countries to increase the authorised capital of the Bank from $10 billion to $21 billion.

The Report records the usual wide variety of Bank activity in fields other than lending. It mentions the negotiation by President Black of a settlement between the United Kingdom and the United Arab Republic, and reports the progress made in mediation of the dispute between India and Pakistan over the sharing of the waters of the Indus Basin.

During the year 30 loans were made to 19 countries, three in countries where the Bank had not previously lent—Gabon, Malaya and the Sudan. The pattern of lending showed the usual emphasis on the strengthening of basic services. Electric power development was the largest objective, accounting for $294 million of the total. These power loans will assist the installation of almost 2·5 million kW of new generating capacity in 11 member countries: Austria, Brazil, Ceylon, Colombia, Denmark, El Salvador, Honduras, India, Italy, Japan and Malaya. In this connection reference is made to Kariba and Ybbs-Persenbeug schemes which have already been described in WATER POWER.

Swedish Turbine Manufacturer's Report

ON May 28, a general meeting of A.B. Karlstads Mekaniska Werkstad was held in Karlstad, Sweden and the annual report for 1958 was presented. This showed that during 1958 the orders received amounted to 56 million kroner, of which 20 million were for water turbines and 7 million for the KMW propellers. The investments for 1958 totalled to more then 8 million kroner part of which was utilised for the construction of the final stages of their new water turbine laboratory.

It was also noted that the Swedish State Power Board, Kungl. Vattenfallsstyrelsen, which is the largest power company in Sweden, is celebrating its 50th anniversary this year. During all these years KMW has often supplied this company with water turbines even as early as 1907 when they supplied two turbines for the Power Board's first station, Olide at Trollhättan. Since then, KMW have made many turbine deliveries to major power plants owned by the Board. As present figures show, KMW-made turbines are developing more than 2 million horse-power for Sweden's largest power supplier.

The second of the three KMW runners for the Kariba power plant in Rhodesia is now ready for despatch. In November of last year the first runner left the works at Kristinehamn for Gothenburg, whence it was shipped to Beira in British South-East Africa. The Kariba turbines are of Francis type, each for 142,000 h.p. at 94·5 m. head and a speed of 167 r.p.m.

Mexican Power in 1958

A REPORT by the *Camara Nacional de Electricidad* states that Mexico had a generating capacity of 2·6 million kW in 1958, an increase of 12·8% over the preceding year. The electric power was distributed among approximately 2,700 communities with a population of 11·8 million, or about one-third of the entire population of the country. The remaining two-thirds have no electric supply. It was further stated that 2,623 plants were in operation last year of which 2,140 were privately owned. 45·3% of the output came from hydro-electric stations and the remainder from thermo-electric plants.

New Zealand Hydro-Electric Investigations

AN extensive programme of hydro-electric power investigation will be put under way by the New Zealand Government this year. It will include the examination of new power-station sites on several North and South Island rivers. The work is estimated to cost approximately £3 million.

The cost will be spread over the next four years, stated Mr. Hugh Watt, the Minister of Electricity. The investigations are complementary to preliminary exploration already made of the power potential of the rivers under consideration. The information to be obtained, added Mr. Watt, will be of considerable assistance to the Government in estimating the power and energy available from the proposed sites on these rivers, and also in assessing the development costs. Provision has also been made for the expenditure of nearly £300,000 to continue investigations on the Wanganui River (North Island) during the next few years.

New Record for Norwegian Developments

THIS year will mark a new record in the harnessing of hydro-electric power in Norway; the total capacity will by the end of the year have increased by 612 MW from 5,400 MW. to 6,012 MW. No less than seven projects are coming into commission this year, in part or in full. The largest is the Övre Vinstra with a capacity of 130 MW. A great number of schemes are under way, however, and in the autumn of 1962 Norway's largest project at Tokke will start the first of its four turbines, each with a capacity of 100 MW. It is expected to be completed in 1963. Other great developments are Nea in the county of South Tröndelag and Tunnsjö in North Tröndelag. £40 million will be invested in the harnessing of water power in Norway this year.

Hamilton River Scheme, Labrador

MINING interests in Labrador are proposing to form a subsidiary company to develop the power potential of the Twin Falls on the Hamilton river. This was announced recently by Mr. B. C. Gardner, chairman of the British Newfoundland Corporation Limited, who added that anything between 50,000 and 300,000 h.p. could be developed at this site. The capacity of the plant initially installed would naturally depend on the demand at the time when it was possible to begin construction but Mr. Gardner said that the Wabush Iron Company had confirmed its intention to take power from the plant. The British Newfoundland Exploration Company and its subsidiaries would also be likely consumers as their prospecting geologists had already revealed a number of deposits containing asbestos and copper in Labrador and copper, lead and zinc in Newfoundland.

Deepening a Water Channel

An account is given of the method adopted of drilling and blasting submerged holes between Lake Victoria and the Owen Falls hydro-electric scheme in Uganda

By ALAN GREGORY, A.C.S.M., A.M.I.M.M.

IN 1954, the Owen Falls dam and power station were completed at Jinja for the Uganda Electricity Board. About a mile upstream of the dam are the Ripon Falls, which were submerged about 8 ft. when the water was impounded. As was expected, the falls restrict the flow of water and cause a head loss of some two to three feet at certain times of the day.

As the load at Owen Falls increased it became necessary to excavate the main channel to an effective depth of 20 ft. at the falls and to cut two other channels of the same depth about ¼ mile farther south, involving the removal of about 10,000 cu. yards and 50,000 cu. yards respectively. The general layout is shown in Fig. 1.

The consulting engineers for this project are Sir Alexander Gibb & Partners, with Stirling-Astaldi (Uganda) Ltd. as the contractors. For various reasons the two southernmost channels have been commenced first and here the depth of water varies between 4 ft. and 16 ft. and its speed between three and eight knots, according to the demand of the power station.

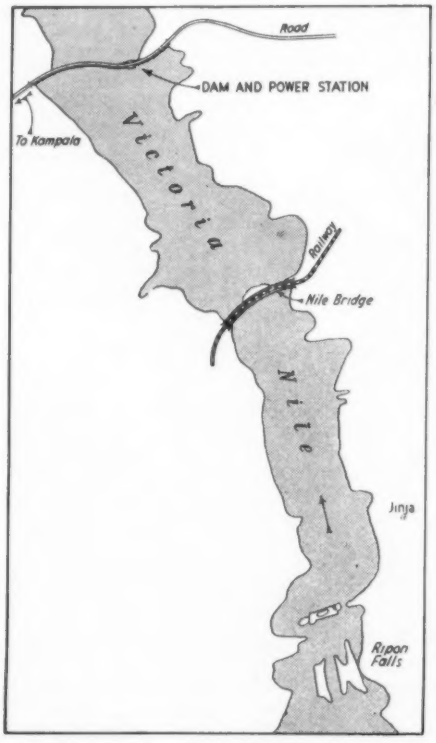


Fig. 1. Map showing the location of Ripon Falls in relation to Owen Falls power station

After considerable thought and some tests with various methods it was decided to build a pontoon and on this to mount rock drills capable of drilling 3 in. diameter holes under water, blast, and then excavate the broken material with a floating grab. The rapid flow prohibited divers from drilling under water, and although a caisson was tried for this purpose, it was not found satisfactory. Furthermore, there is considerable danger from bilharzia in the water itself. To complicate matters still further the rock is very fissured and is overlain by large loose boulders. Finally, it was decided to use six Holman 3½ in. Rotofeed drifters and Avesta extension drilling equipment (1 in. hollow hexagon rods with 1¼ in. lugged shank) together with one Holman T60R (530 cu. ft. per min.) compressor and one older compressor of another make to supply the air. The six drifters are mounted on two parallel H-section horizontal girders on a double-unit barge or pontoon, the position of which is controlled by wire ropes anchored from either side of the river and attached to the pontoon by two Holman winches. In view of its length the cable is suspended on the surface of the river by anchored empty oil drums, this method giving a steadier pull and preventing the fouling of rocks on the bottom. The compressors, mounted on the shore, supply the air through a 6 in. diameter steel pipe, also supported on oil drums, which is connected to both the shore and the pontoon by means of a flexible rubber hose of similar size.

The grabs used for the excavation work are of the Ruston Bucyrus 33 RB type and are also mounted on a pontoon; in addition a 54 RB is mounted on a small island near which blasting has recently taken place. Where complete removal from the river of the blasted rock is not necessary, the grab is used to pile it on to the island; otherwise 30-ton barges are used to take it completely away. The barges are loaded by the 33 RB and propelled by wire ropes anchored to the barge and operated from the shore by a Holman electric winch, unloading being effected by another grab situated on the shore.

Owing to the unsteadiness of the pontoon for drilling and the variable nature of the blue amphibolite rock, the following procedure has been adopted. The drifters themselves are mounted on aluminium-alloy feed cradles which are held in position by means of a swivel clamp on each of the two parallel horizontal girders. These girders are bolted to the end of the pontoon and overhang the water. This double clamp, where normally only one is used, prevents any possibility of movement of the machines due to a momentary drift of the pontoon. The height of the drifters above water level, when fully extended to the limit of the cradle, is about 2 ft. Below and in line with each drifter two jaw clamps are secured, capable of being opened to about 3½ in.

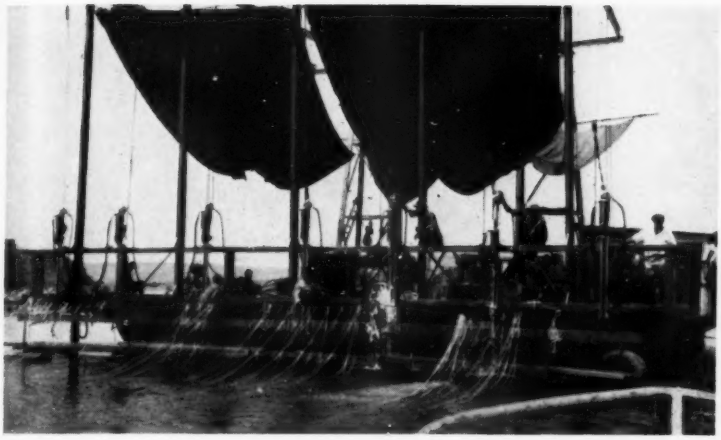


Fig. 2. The drilling pontoon in use. Note the sun screens and the cordtex fuse

Having plumbed the water depth under each machine, a suitable length of 3 in. mild-steel pipe is chosen for each machine, lowered to contact the rock and clamped tightly. An Avesta $2\frac{7}{8}$ in. bit, attached to a 3 ft. Avesta 1 in. hollow hexagon threaded extension steel is then inserted and coupled to the drifter by a $1\frac{1}{4}$ in. lugged shank with water swivel attachment. Through this latter air is blown instead of water, which is found very effective in clearing the chippings. Water was tried originally but was by no means so effective. The water swivel attachment method of passing the air allows more air to enter the cutting face than by the more usual method of passing water through the machine.

Drilling now proceeds for about 6 in. when the rods and pipe are extracted and a 3 in. pipe of the same length as the first, but with a rough thread and taper on one end, is inserted and screwed into the 6 in. deep hole. Through this pipe drilling now proceeds but with a $2\frac{1}{2}$ in. cross bit substituted for the $2\frac{7}{8}$ in. bit. Extension steels 6 ft. in length are now used instead of of the 3 ft. steels wherever possible. When the hole reaches 18 ft. from water level the equipment is removed and the hole charged.

As far as possible the six drifters operate at the same time, but in any case all the holes are charged together. For this, 74% gelignite in 22 in. by 2 in. sticks are used, cordtex detonating fuse being inserted with the first cartridge. The end is sealed with an empty detonator cap and covered with pitch. The six fuses are bunched together after loading is completed, tied, and attached to a float.

The pontoon next retreats away from this line of holes and a further five lines are drilled and charged, making 42 holes in all. When ready for blasting, each line is individually connected up in parallel but connected in series with the next line until the last is completed. Each line has a 17 millisecond delay detonator between it and the next line, the initiation of the blast being by ordinary safety fuse attached to the cordtex. The holes are spaced at 7 ft. centres, with 6 ft. between each line. Approximately 1,000 cu. yards are broken in one blast and an efficiency figure of about $1\frac{1}{2}$ lb. per cu. yard is obtained, but this is kept low because of the virtual impossibility of redrilling the ground should satisfactory fragmentation not take place. Furthermore, the adjacent rock was apt to be disturbed by the blast if a larger amount is broken at the same time.

The time cycle from one blast to the next averages three days, this being due to the class of labour available, misalignment of the holes caused by unevenness of the surface, and dropping of tools in the water. Other causes of loss, are the attempted destruction, possibly by curiosity, of the fuse floats by hippos in the water at night, which has been partly overcome by having a good night-watchman to scare them off.

Before a blast, it is amusing to watch the Africans in dug-out canoes awaiting the explosion from the nearest bank. As soon as the flying rocks have settled and often before, each African can be seen paddling furiously into mid-stream to catch the many fish which are killed or stunned. Once out into the current the boats become almost uncontrollable and one wonders why no one is drowned. The fish themselves vary from 1 to 4 lb. in weight and find a good market locally.

In this brief description of the operation it will no doubt be appreciated that before any actual work was started and a satisfactory cycle of operations decided upon, a considerable amount of planning and experimenting had to be done. The writer wishes to acknowledge the kind help he received from the staff of Stirling-Astaldi (Uganda) Ltd., and Sir Alexander Gibb & Partners (Africa). He also acknowledges his indebtedness to the Uganda Electricity Board for permission to publish this article.

Fig. 3. Front of pontoon showing the Holman SL280 drills

The IAHR Montreal Congress

A series reporting the proceedings of the International Association for Hydraulic Research Eighth Congress held in Montreal, Canada, in August 1959. Many of the papers reviewed are likely to be of particular interest to hydro-electric engineers and to the designers of hydraulic structures

PART ONE

SOME 300 delegates from 28 countries attended the Eighth Congress of the International Association for Hydraulic Research in Montreal held in August, to discuss over 80 papers on selected topics of importance to hydro-electric engineers, including two seminars for which informal notes were submitted. This report deals briefly with formal meetings on: A. Gates and Valves; B. Navigation Locks; C. Air Entrainment, Air Shafts and Valves; and D. Density Currents; and with seminars on: Ice Problems in Hydraulic Structures and Equipments; and Transportation of Materials in Water.

As might be expected, many papers related to significant geographical problems or to special aspects of major projects which have been in the news for technical, political or economic reasons, such as the St. Lawrence Seaway, Netherlands Waterways, Rance Tidal Power Scheme and the Aswan High Dam.

SUMMARY ON GATES AND VALVES

Radial Gates

One group paper originating from the Netherlands marked important advances on scale effects in gate structures subject to rapidly changing loads. Schoemaker (Paper No. 30A) summarised more detailed contributions devoted to a 56·6 m. radial discharge sluice gate where variable hydrodynamic forces are exerted on movable and elastically deformable structures exposed to storm wave action due to:—

(*a*) Hydrostatic pressures.
(*b*) Varying positive and suction pressures caused by currents and eddies.
(*c*) Slowly changing surface wave pressures.
(*d*) Wave impacts.

Model tests were used to determine the gate leaf shape and the forces, pressures and dynamic effects with the flexibility, torsional rigidity and mass distribution to the proper scale for vibration created by:—

(i) Pulsating narrow frequency band forces coinciding with natural structural frequency with movement feedback to turbulence causing periodical forces.
(ii) Random wide frequency band pulsations with erratic structural response. This seems to be influenced by side walls where pressure fluctuations are 1·5-2·0 times those in mid-section.

Damping is an intricate response characteristic and is reproduced only when the model and prototype move in the same way. Although tests showed that water damping predominates, internal damping may cause distorted response characteristics especially where coupled vibrations can occur and must be considered because the vibration build-up excited by sudden random load fluctuations and maximum deflections depend on it.

Because self-sustained vibrations due to fluctuating forces occur when a gate is slightly raised, investigations were made to ascertain whether they correspond with low natural structural and hoisting equipment frequencies. The recorded deflections, analysed on the electrical filter concept of a mass spring system with random load wide frequency band inputs, disclosed sinusoidal free vibrations with low actual/critical damping ratios. Other tests showed that:—

(i) Wind generated and regular waves produced different pressure variations.
(ii) With short duration impacts the compressibility of water containing air bubbles affected superposed pressure fluctuation peaks.

Companion papers amplified these points:—

Paper No. 25A (*Aartsen, Bijker & Van Heemskerk*) concerned the shape of the discharge sluices, including the seawardside approach floor. Many alternative designs were studied before a satisfactory arrangement was found. Floating ice demanded wide stream channels to pass 20,000 cu. m. per sec. and 17-56·6 m. openings were selected with 4·5/3·0 m. water level differences acting from or towards the sea. One radial gate is anchored on each side of a prestressed concrete bridge and preliminary tests showed that critical impact pressures of 40 tons/m² could develop in 0·01 sec.

Paper No. 22A (*Aartsen & Venis*) described wind tunnel wave investigations to trace dynamic pressures on seaward inclined concave surfaces exposed to wave attack of given spectrum and obliquity from which critical design conditions for swell, wind and gust velocity, water level and wave period were determined. Wave impact may occur at any point over a width of twice the gate height depending on wave crest length and approach floor shape, the superimposed dynamic component being approximated by exponential functions.

The twin double plated radial gates must move through sheet ice and to determine underflow suction and tangential forces when partially raised, *Dirkzwager & Prins* (*Paper No. 24A*) used a model incorporating the hoisting mechanism elasticity and gate mass distribution to study water depth, wave height and period effects for various outer and inner gate openings.

Paper No. 27A (*Hart & Prins*) analysed turbulent pressure fluctuations and bottom beam movement interaction where limited water transmitted energy may, at resonance, generate large structural amplitudes.

The random nature of turbulence seems to require statistical correlation of forces and free amplitude time curves.

Paper No. 23A (*Allersma*). The dynamic behaviour of the discharge sluices with underflow and wave forces required data on the influence of water surrounding partially submerged gates and model measurements were made to derive design coefficients, using the virtual mass principle to give the equivalent of breaking waves.

Vertical Lift Gates

GATE FORCES AND VIBRATION

Paper No. 5A (*Grzywienski, Austria*) mentioned that studies and model tests concentrated on flow effects in single and multiple leaf spillway and deep seated gates have resulted in improved structural designs relating to skinplate, seal and girder location and reduced hydraulic forces, giving freedom from vibrations with cheaper operating equipment.

Paper No. 26A (*Bryce & Foulds, Canada*) provided an excellent comparative model study of eight types for the Niagara and St. Lawrence Seaway projects to ascertain whether downpull, which can be large relative to gate weight and uplift, which may prevent closure, can be measured for fixing hoist capacity and whether design modifications are beneficial. From these tests, under different operating conditions, they concluded that a general analytical solution required further work.

Paper No. 16A (*Selim & Duport, Egypt/France*) was devoted to factors affecting operating forces on the diversion tunnel discharge regulation gates at Aswan high dam, in relation to the seal position, bottom bevel, grooves and flow characteristics for free or drowned discharge.

Another important contribution came from *Abelev* (*Paper No. 10A, U.S.S.R.*) who dealt with the theoretical and experimental aspects of pulsating hydrodynamic loads on bottom outlet sliding gates. The investigations showed that non-periodic oscillations comprised positive and negative impulses of different form, magnitude and duration dependent on head, hydraulic conditions and the vortex zone behind the gate. Gate vibration phenomena depend on pulsating loads, structural rigidity of the system supports, internal and external forces, non-elastic resistance, gate opening, the downstream conduit regime and air demand for the conditions where the jump roller:—

(i) Covers the gate bottom.
(ii) Moves away from the gate but remains inside the conduit.
(iii) Is outside the conduit giving free air access.

The paper covers:—

(*a*) Experimental research into hydrodynamic exciting forces, pulsating frequencies and amplitude for variable conditions.
(*b*) Parameters characterising gate vibration based on induced oscillation theory.

The vibration spectrum and dominant frequency were determined by a statistical analysis of strain gauge records on three scale models confirming that the dominant pulsation:—

(i) Amplitude varies as the cube, and
(ii) Frequency varies inversely as the square root of the scale coefficient.

Paper No. 8A (*Thomas, U.S.A.*) gave practical reminders of the relationship between model and prototype experience in a report on the "Hydraulic Performance of 96 in. Regulating Gates at Shasta Dam." These operate at heads exceeding 200 ft. and measurements included pressure, discharge, air demand, bonnet vibration frequency and amplitude for various heads and openings, from which the following points emerged:—

(i) Sub-atmospheric pressures were more severe than forecast by model tests but insufficient to produce cavitation.
(ii) Gate groove pressures were not excessive indicating that jet impingement was absent.
(iii) Model tests closely predicted discharge.

An unusual aerodynamic study, in Froude similitude, was reported by Maggiolo (*Paper No. 1A, Uruguay*) to determine the effect of wind on placing emergency bulkheads in their grooves. Torsional and lateral oscillations were examined in relation to the suspension system, crane and building location, water level, wind speed and direction for velocities up to 50 m.p.h.

FUNDAMENTAL CONSIDERATIONS

Several papers by Egyptian and French authors concerned basic problems on high-head gates for the Aswan High Dam diversion tunnels which must operate partially open for long periods.

Paper No. 17A (*Zaky/St. Marc*) summarised experimental work undertaken to improve jet forms and to define the hydrodynamic forces involved and their effects on gates and grooves. Electric-acoustic methods were employed for defining jet profile and dye injections revealed groove vortices, the strength of which depended on head and gate opening.

Paper No. 18A (*Helmy/Lemoine*) utilised flow nets and equal pressure/velocity lines to study partial gate opening pressure distribution and groove effects, from which it appears that:—

(*a*) Flow retardation upstream produces separation or a top roller at the tunnel ceiling.
(*b*) Along the centre line the top roller rotates almost symmetrically but at the side walls it tends to follow the flow streamlines.
(*c*) Pressure gradients near the gate bottom are steeper than at the top where the velocity variations are more rapid because there is less flow inertia. The jet spread is therefore greater than at the bottom until normal flow is established when the static head causes it to widen at the bottom.
(*d*) The jet spread should clear the grooves to avoid vortices and cavitation in them.

Paper No. 19A (*Wafa/Advani*) examined the flow across a gate groove by extending Tollmien's mixing theory to three dimensions to explain free and forced slot vortices. There is an unstable surface at the velocity discontinuity separating a jet from the adjacent dead water where turbulence tends to equalise the velocity and diffusion spreads the mixing zone downstream. In three dimensions it appears that water is sucked into and pumped out of the grooves and a revolving core produces an enveloping helical flow with continuous mass transfer between the periphery and the centre thus causing vortices, depressions and possibly cavitation. To minimise mixing, the grooves should be square rather than rectangular with the downstream ends set back so that the leading and trailing corners define an appropriate flow line.

Cavitation Tests. Paper No. 21A (*Duport & Bertrand, France*) defined the limiting conditions for gate cavitation tests under submerged free jet or drowned

jump conditions where the complex parameters depend, inter alia, on upstream and downstream conduit head losses. The importance of Froude similitude is stressed.

Special Gates

SEMICIRCULAR GATES

Paper No. 29A (*Kolkman, Netherlands*) described 1:6 and 1:20 Froude scale tests on a 48 m. semicircular hinged gate to ascertain vibration characteristics with underflow when partially raised. Elastic, flexural, inertia, and torsional rigidity properties were embodied in the low elastic modulus, hardened polyvinyl chloride model for which natural vibration modes were calculated, including the virtual vertical and horizontal water mass. The tests revealed that:—

(i) Lower girder web perforations are important for reducing vertical forces.
(ii) Vertical vibrations occur near the hinges.
(iii) Measured deflections for comparing stress computations and structural modifications require scale effects data, because:—
  (*a*) Exciting forces and mutual displacements may create self-sustained vibration.
  (*b*) Water and internal damping are important.
(iv) Within the experimental limits of both models:—
  (*a*) The water is turbulent and water damping forces have the same magnitude.
  (*b*) Internal damping effects on vibration amplitude are small.
  (*c*) The exciting forces and the order of magnitude of current stimulated vibrations are reproduced.

SUBMERSIBLE SECTOR GATES

Paper No. 7A (*Hellstrom, Sweden*) outlined model tests to determine the hydraulic characteristics and forces for differently shaped fully submerged sector gates designed to pass maximum floods and timber with minimum water consumption during low flow periods.

NAVIGATION LOCK VALVES

Paper No. 13A (*Nelson, Johnson & Fidelman, U.S.A.*) described studies to evaluate the dynamic effects of water flow over and under submerged sector valves controlling culvert discharge during locking operations. The initial opening force when the top seal is broken depends on the gap and is approximately three times the static hydraulic load and about two-thirds of the hoist capacity which suggest that blade design justifies close examination.

GATES FOR TIDAL POWER DEVELOPMENT

Papers Nos. 11A and 12A from France (*Kammerlocher, Daumy, Bertrand & Duport*) introduced special purpose butterfly valves which may have other applications.

Butterfly Valve with Adjustable Horizontal Elements

For the experimental double flow bulb generating unit installed in a disused lock at St. Malo a three element valve was designed to pass 350 cu. m. per sec. for accurate water level control and emergency use. The horizontal lenticular shaped blades are inclined at 65° when closed and are 12 m. long by 3·7 m. wide by 1·0 m. thick and operated by individual oil pressure servomotors.

A 1:40 model covering the whole layout including the harbour, lock and the valve was used for a general study of water level regulation for basin/sea and flow for sea/basin working. The 1:20 valve studies included:—

(i) Blade hydrodynamic forces.
(ii) Flow lines near the blades.
(iii) Operating positions in both flow directions.
(iv) Level/discharge/blade position characteristics for basin/sea working.
(v) Minimum turbine inlet levels for sea/basin working to prevent air entrainment.
(vi) Basin/sea flow stability by nappe air admission to avoid separation or overflow at partial openings.
(vii) Flutter due to coupling of hydrodynamic forces. Theordsen's theory confirmed that the incipient instability velocity exceeded the maximum water speed.

Butterfly Gates, between the sea and the Rance tidal basin to accelerate level equalising preparatory to commencing operation. Operating forces relative to a normal blade were reduced by vertically pivoted 20° helicoidal blades for which the torques were 190 and 98·5 m. tons respectively. For economy in operating gear costs the blades are set in pairs in a Venturi opening and controlled by twin opposed piston servomotors and separated by a pier to reduce forces transferred to the concrete in either flow direction.

The principal details are:—

Section	8 m. by 7·5 m.
Discharge	400 cu. m. per sec. under 1 m. head
Maximum Heads	Opening 5 metres
	Closing 13·5 metres

The design was governed by cavitation limits and discharge characteristics and 1:30 Froude similitude model tests permitted determination of:—

(*a*) Profile improvements and optimum openings by photographic recording of cavitation pockets.
(*b*) Cavitation threshold by visual, acoustic and electronic stethoscope observations so that the importance of imperfections on the blades and concrete surfaces could be assessed.

RECTANGULAR PIVOTED GATE

A rectangular gate pivoted horizontally and free to rotate in the flow direction was discussed theoretically in *Paper No. 20A* (*Geffen, Israel*) relative to upstream and downstream depths, head loss and pivot position. The experimental work disclosed critical angles and instability at large Froude Numbers caused by an upstream wave train interacting with a downstream roller. The gate provides a simple open channel flow control but quantitative agreement between theory and experiment needs investigation.

Gate Operating Problems

SINKING SECTOR GATES

Paper No. 15A (*Petrikat, Germany*) explained that accurate automatic control demands sufficient differential head under flood conditions, preferably without pumping, and freedom from elasticity due to air in the upper part of the sector In new designs these difficulties are overcome by:—

(i) A 2·35 m. diameter hemisphere anchored above the hinge where the high velocity water produces ample chamber/outlet pressure differentials for lowering.
(ii) A steel plate forming a water or concrete filled cell so that no air is trapped in the upper part.
(iii) Special valves and sequential control mechanisms which allow presetting within ± 12·5 mm.

SPHERICAL BALL GATE

Paper No. 2A (*Gerodelle, France*) dealt with severe repeated waterhammer and cavitation generated in a

(*Continued on page 414*)

Calgary Power Expansion

In our September and October 1954 issues, pages 324 and 371, we published an account of the hydro-electric power plants which Calgary Power Limited had installed, and the present article gives some particulars of further progress that is being made

THE immediate expansion programme of Calgary Power Limited entails three projects—expansions to the Spray and Rundle hydro plants, the addition of a third unit to the Wabamun steam plant and the development of the Brazeau river water-power development. By 1964 the Company's generating capacity will have been increased by about 365 MW from the present 409 MW. A year ago load growth had dropped to a post-war low of 5·2%, but during the past 12 months the power demand rose steadily until the present annual rate of load increase on the system approaches an average growth of 13·5%.

Spray-Rundle Extensions

The Spray-Rundle extensions will add a further 80 MW to the Company's hydro generating capacity. A second 50 MW will be added to the Spray plant, increasing its capacity to 100 MW, and a 30 MW unit will be added to the Rundle plant, increasing its capacity to 47 MW. Work on these extensions is progressing according to schedule and it is expected that the new units will be operating by October 1960.

The existing Spray development consists of a long canal system leading from the Three Sisters power house with pondage formed at Goat and Whiteman's Pond. The intake structure is at the north-east end of Whiteman's Pond at the top of the pass. A pressure tunnel drops 900 ft. to the power house, which contains a single 62,000 h.p. vertical turbine and generator with the tailrace discharging into the forebay of the Rundle plant.

The extension in hand consists of an additional intake near Whiteman's Pond, a woodstave pipeline, a surge tank, a steel penstock and an addition to the powerhouse to contain a second 62,000 h.p. vertical set. A gross head of 908 ft. will be developed between the intake and the powerhouse—over five times the height of the Niagara Falls.

The penstock will be constructed of "T1" steel which is stronger than the steel that is usually used on this work. A thinner gauge can thus be used, reducing the tonnage, and consequently the overall cost is lower, despite the need for elaborately controlled welding procedures. The intake pipe, woodstave pipeline and steel penstock will be 2,400 ft. in length.

The existing Rundle plant was completed in 1951 as an integral part of the Spray project. Rundle uses the discharge of the Spray plant, through the 315 ft. head which exists between the Spray tailwater and the Bow river. The rating of the existing unit is 17 MW. The Rundle extension will be carried out concurrently with the extension to the Spray plant. A new intake is being constructed and a new steel penstock which will be 1,380 ft. long. There will be no surge tank or turbine shut-off valve. The new 30 MW unit will be housed in an extension of the existing power house.

Wabamun III

The Wabamun steam plant, located on the shores of Lake Wabamun, 42 miles west of Edmonton, has at present a generating capacity of 144 MW. The first unit was commissioned in the autumn of 1956, and the second unit, a duplicate of the first, was commissioned in 1958. A third 150 MW unit is now scheduled for 1962, and will burn coal, the plant being adjacent to a 60 million ton deposit of strippable coal. The plant is at present burning natural gas, but the

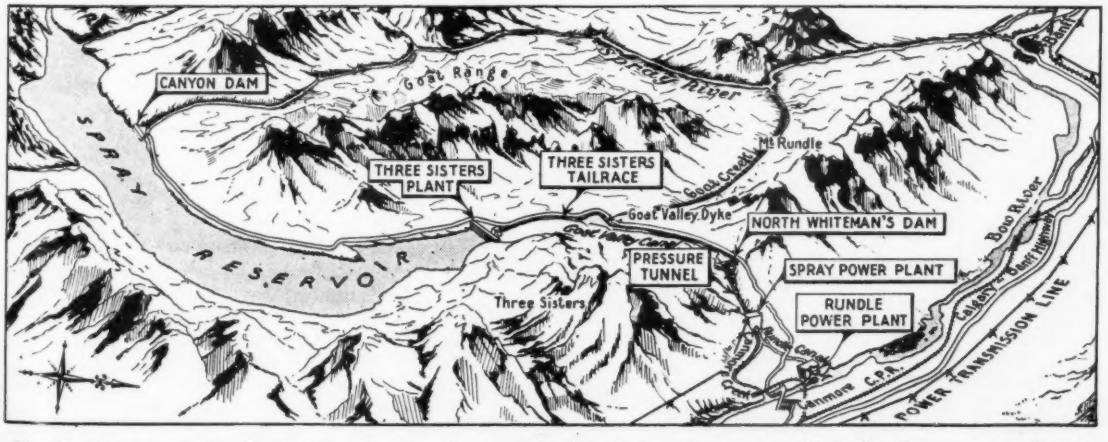


Fig. 1. Diagrammatic illustration showing the topography of the country in which the Spray river plant is situated. Extensions are projected for both the Spray and Rundle plants

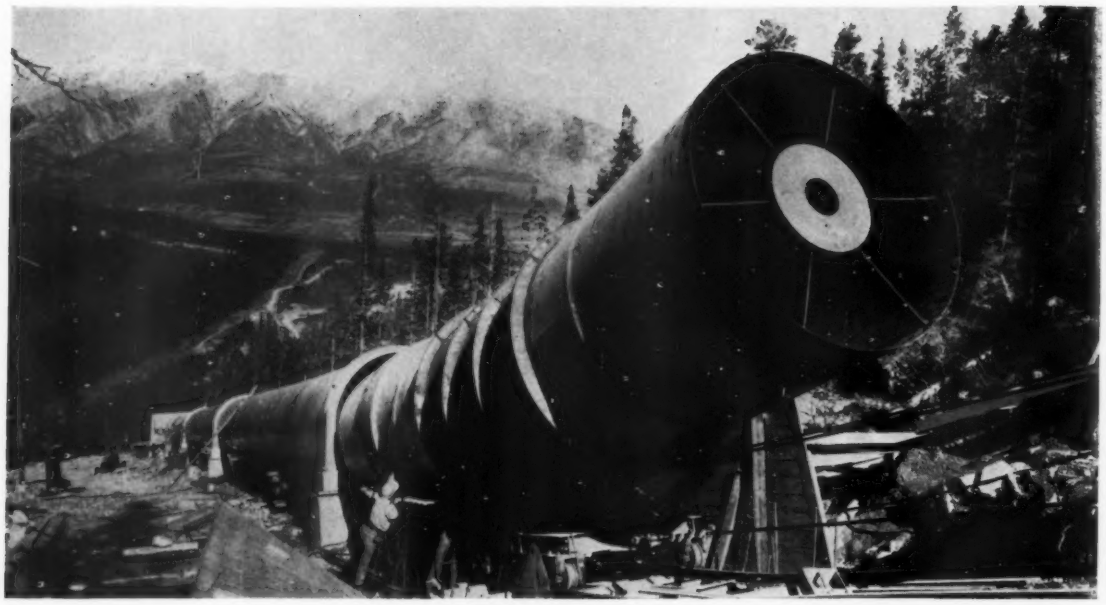


Fig. 2. Placing a 15 ton bend section of $\frac{5}{16}$ in. T.1. steel penstock for the expansion of the Spray project

Fig. 3. View of Big Bend site on Brazeau river looking upstream from site of main dam

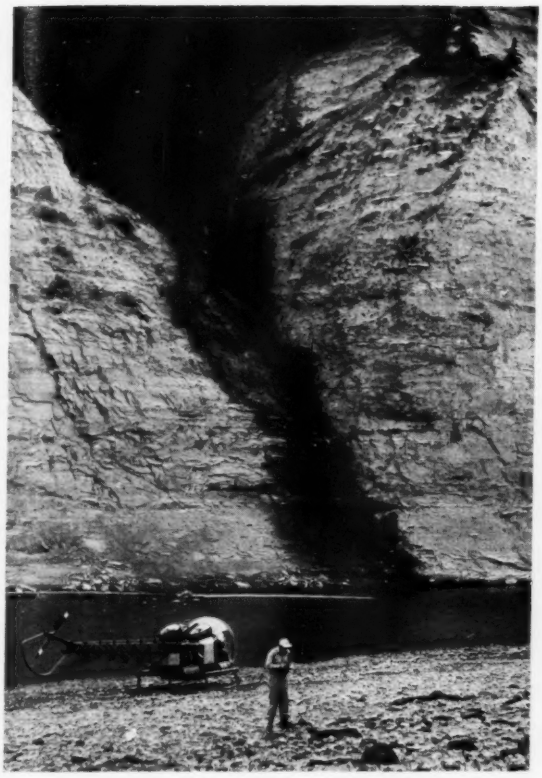


Fig. 4. Brazeau power development. Helicopters were used extensively for the preliminary survey work

Fig. 5. Downstream view from main dam site Brazeau river showing rugged nature of the country

coal represents a low-cost, long-term fuel supply for this plant.

Brazeau River Development

The Brazeau hydro power development will cost some $45 million, take five years to complete, and will have an initial capacity of 150 MW. Initial construction, now under way, will enable some power to be available in 1963 should this be required to meet the high load growth at present being experienced. Final designs have not been made and planning is still in the preliminary stages so that no contracts have yet been let.

The Brazeau site is located on the "big bend" of the river, 96 miles south-west of Edmonton. The site on this tributary of the North Saskatchewan is in the heart of heavy forest and muskeg country. Over the past two years company engineers have been carrying out preliminary investigations, including stream-flow studies, topographical surveys, drilling and soil testing. Much work has still to be done in regard to testing the soils to be used in construction and in the foundation conditions that have been encountered.

Initial development and construction of the Big Bend site will include:—

(*a*) a 10 million cu. yd. earth-fill dam two miles long;
(*b*) a reservoir in the Brazeau valley which will contain 900,000 acre-ft. of usable storage, having an area of 37 sq. miles;
(*c*) a 12 mile long canal below the dam which will convey the water to the power house on the Brazeau river about 6 miles west of the junction with the North Saskatchewan;
(*d*) a pumping station at the main dam to pump water from the reservoir to the canal;
(*e*) a concrete spillway adjacent to the pumping station;
(*f*) a power house that will contain initially a 200,000 h.p. generating unit.

A head of 400 ft. will be developed and three or more additional units can be added to plant. As the reservoir will not be filled completely by the fall of 1963, there will only be a limited amount of water available during the winter of that year, but it is anticipated that the reservoir will be full during the following winter and that it will then be possible to increase the flow of the North Saskatchewan to a substantial extent.

The Brazeau hydro plant will be able to meet the widely variable loads represented by residential and commercial consumers in the villages, towns and cities of Alberta, and in virtue of its storage facilities will help to keep power costs down. The project, in fact, is one of the keys to the continued production of low-cost power. This type of installation, operated in conjunction with thermal plants, such as the Wabamun plant, which produce some of the lowest-cost base load energy in North America, will assure Alberta power consumers of an adequate supply of low-cost power at all times.

When the initial construction and development

Fig. 6. Bulldozed trail through Tamarack swamp at Big Bend site, indicating difficult nature of approach. This road could only be used when frozen

work is completed it will be possible to study more closely what the ultimate capacity of the plant will be. The initial cost of the project will be high, but generating units can be added at a relatively low cost per kW.

The present rapid annual rate of load increase on the company's system is due to greater industrial activity and a general expansion in the demand for power by residential and commercial consumers. For instance, the Inland Cement Company have recently announced a $5 million expansion programme for their Edmonton and Regina plants. At the Edmonton plant a third kiln and related facilities will be built to increase capacity by an additional 1 million barrels a year. Between January and June 1959, 12 new industries settled in the Calgary area alone, and some $17 million will be spent by these concerns on buildings and equipment. The Brazeau development is intended to meet the need for power created by these and similar industries and by the people who work for them.

From page 410

short pipe 1 m. diameter by a spherical ball intended to close a temporary opening in a dam under 4 m. head. This was overcome by enlarging the outlet forair admission.

WATERHAMMER IN PRESSURE CONDUITS

Paper No. 6A (*Cabelka & Franc, Czechoslovakia*) utilised Schnyder Bergeron diagrams adapted to include gate area characteristics to examine closing laws relative to waterhammer limits. It also drew attention to turbine characteristics in particular applications.

AUTOMATIC FIXED ROLLER GATES

Paper No. 3A (*Gerodelle, France*) outlined how the automatic operation of weir crest, counterweighted float controlled, fixed roller gates was disturbed by water rising through the aeration gap to exert pressure on the horizontal beam. Web perforations were ineffective but satisfactory performance resulted from adding a downstream skinplate.

SECTOR GATES WITH SILT LADEN WATER

Paper No. 4A (*Olymieff, France*). In an automatic level control sector gate installation the two laterals were counterweight float operated and the two central ones by winches via floats. Although the system was stable, the former opened too late and the latter too early, resulting in slight over-elevation. Investigation revealed that the water feeding the low level counterweight float wells was heavily charged with silt and shallow well intakes in a region free from level fluctuations following gate movements restored accurate control.

(*To be continued*)

Sandvik Expands Research Facilities

The Sandvik Steel Works Company Limited, Sweden, have decided to expand their existing laboratory facilities to meet the need for more intensified research and development work on their high-quality steel products. This is to be achieved by the construction, at an estimated cost of £265,000, of a new five-storey building which will adjoin the Company's main steel laboratory. The new building, which is scheduled for completion in 1961, will increase the existing main laboratory floor space from about 32,000 sq. ft. to approximately 70,000 sq. ft. and will contain the most modern equipment available. At present some 250 scientists and technicians are engaged on research and development work at the main steel laboratory.

Hydro-Electric Potential in Sweden

Sweden's gross mean hydro-electric potential has been computed by two methods, recommended by the ECE, and the results are compared

By THURE BERGLUND and SVEN OLOF LARSSON
of the Swedish State Power Board

FOR obvious reasons the economically exploitable hydraulic power resources of Sweden have been the subject of great interest. Many inventories have been made in the course of years and each fresh estimate has given a figure higher than the one preceding it. The first stocktaking was recorded in 1923 and arrived at an annual electricity production of 32·5 TWh (1 TWh=10^9 kWh). The last inventory was made in 1954, the result reached being about 80 TWh per annum. Thus, in the course of three decades, the hydraulic power resources regarded as economically exploitable had increased by approximately 150%. The reason for this is to be found both in technical progress and in the more complete statistics available regarding water supplies in the rivers.

Natural resources in hydraulic power have also been the object of computation. In "Sveriges Vattenkrafttillgångar"—Sweden's Hydraulic Power Resources (Bulletins of the State Meteorological Hydrographical Office[1], volume 5, No. 6), by Ragnar Melin, published 1930—there are reported resources equivalent to 15·86 million kW annual mean output at 100% efficiency, which would give an annual production of 139 TWh. Taking also the smaller waterfalls into consideration Melin has later in "Atlas över Sverige," published in 1957, calculated the natural resources at 17·82 million kW corresponding to an annual production of 156 TWh.

In view of the fact that the statistics available in recent years have given higher figures, it would seem probable that the resources in natural energy as yet recorded are too low. It has, therefore, been found desirable to carry out a fresh estimate of these resources. The necessity has been made even more apparent by the work that is being done by the Economic Commission for Europe (ECE), Committee on Electric Power, directed towards making a comparative inventory of the resources in the countries of Europe. Thus, the method of computation applied in the present inventory is that recommended by ECE. The computation comprises both the annual value of natural resources and their distribution over the winter and summer half-years.

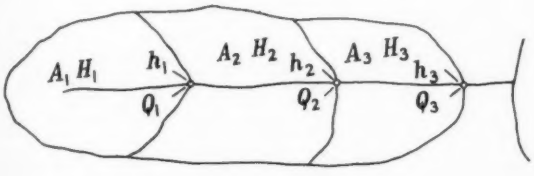


Fig. 1. Diagrammatic river catchment

[1] The State Meteorological and Hydrographical Office has since been reorganised as the Meteorological and Hydrographical Institute, abridged to SMHI.

Method of Computation

In estimating Sweden's gross hydro-electric potential two methods of computation have been employed, both of which are described below.

Method I. In Fig. 1 the catchment area of a river is divided into three surface sections having the areas A_1, A_2 and A_3 sq. km. The mean altitude amounts in sections to H_1, H_2 and H_3, and the lowest point in each section where the gauging station is located has the base altitude of h_1, h_2 or h_3 metres above sea level. In these gauging stations the mean water discharge amounts to Q_1, Q_2 and Q_3 cu. m. per sec. respectively.

If we assume 100% efficiency, there is obtained a mean output P_m during the period of time covered by the figures, as per the following expression

$$P_m = 9{\cdot}81 \quad [Q_1 \ (H_1 - h_1) + (Q_2 - Q_1) \ (H_2 - h_2) + Q_1 \ (h_1 - h_2) + (Q_3 - Q_2) \ (H_3 - h_3) + Q_2 \ (h_2 - h_3)] \text{ kW.}$$

The mean output or the energy quantity in each surface section is thus reckoned on the basis both of the volume of water arising in the section and the volume of water that has arisen above the gauging station next upstream. The corresponding amount of energy is then obtained by the multiplication of the time in hours and the power supply per sq. km. by division by the area.

Method II. In response to the desire of ECE the gross hydro-electric potential has also been calculated in the following manner. In each area there has been reckoned the mean output P_m according to the following expression:

$$P_m = 9{\cdot}81 \ Q \ H$$

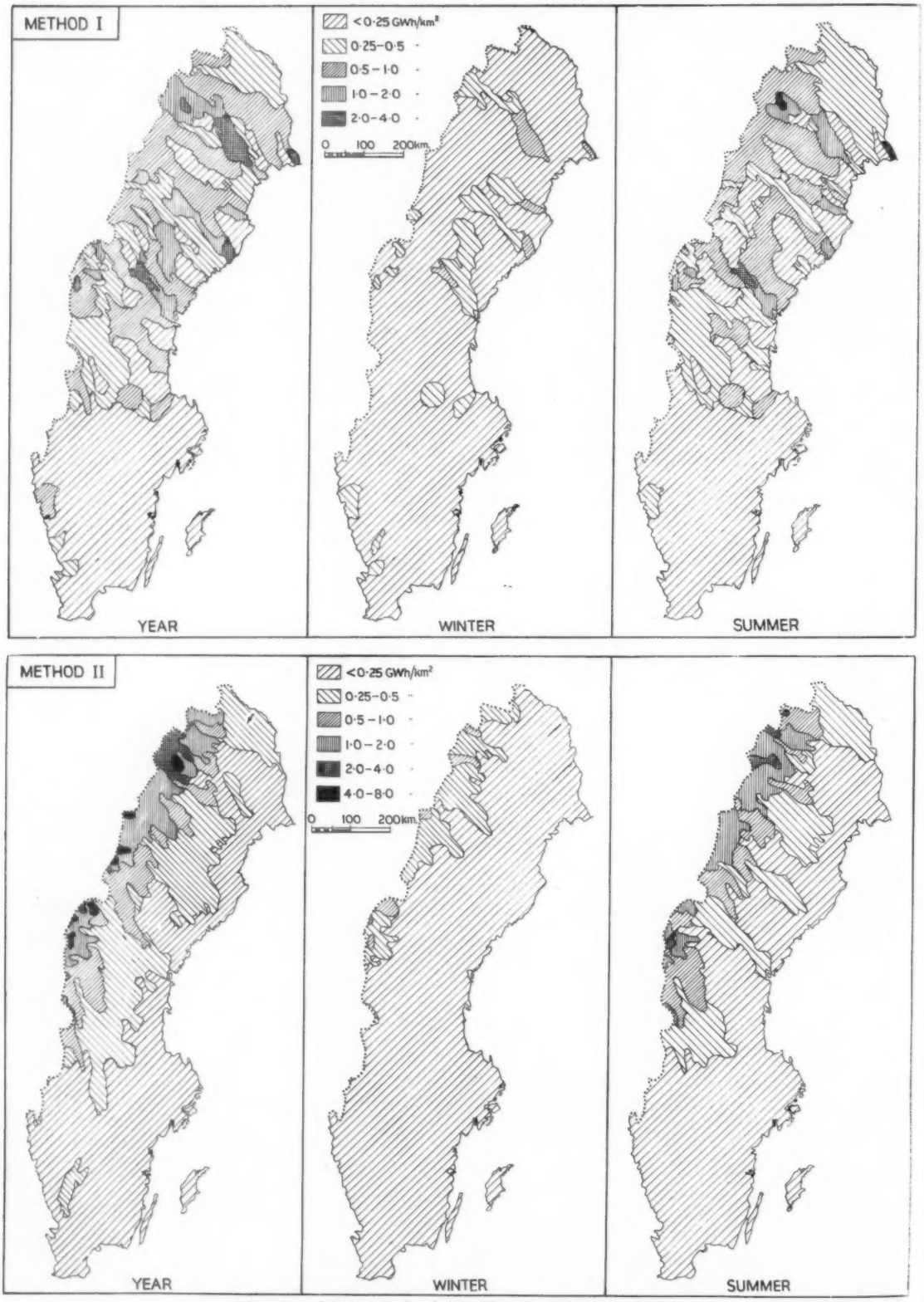
where Q=the water volume delivery in the area concerned in cu. m. per sec. and H=the mean altitude in m.

Requisite Data

As may be seen from the above account of the computation method, there is required a division of the catchment areas of the streams into sections bounded by gauging stations with a known water discharge over a series of years. For this purpose there has been employed the map of the water discharge stations as given in SMHI Bulletins series D No. 6, "Water Discharge in the Rivers of Sweden" by Ragnar Melin. On this map there have been drawn the catchment areas of the different gauging stations.

The water discharge at these gauging stations is given in the book "Water Discharge in the Rivers of Sweden," which also gives the area of the catchment region and the altitude. The altitudes of the gauging stations have been taken from the SMHI Year Book for 1945, III, Hydrology and Hydrography.

The material referred to covers the required particulars for almost all the country. In fact, all that



Figs. 2 and 3. Isopleths derived by alternative methods

is lacking is that part of the catchment areas of the different rivers situated between the lowest gauging station and the sea, and also the surface sections on the islands of Öland and Gotland. These catchments have been drawn in with the aid of topographical maps. The mean altitudes in the area sections thus obtained have been calculated with the aid of these maps, the area having been measured by planimetering, and the water discharge at the river mouth has been calculated with the aid of characteristic figures for precipitation and evaporation and with guidance from the specific flow in adjacent areas. The flow in the different coastal areas differs substantially and shows approximately the following figures:

North Sweden 10 lit. per sec. per sq. km.
South Sweden 5 lit. per sec. per sq. km.
West Sweden 15 lit. per sec. per sq. km.

In the regions bordering on Finland, corrections have been made for those parts of the catchment areas lying in the neighbouring country. Also in the regions bordering on Norway corresponding corrections have been made as far as possible.

The figures for water discharge at the different gauging stations used cover periods ranging from 10 to 100 years, usually 25 to 30 years, but in some isolated cases more than 125 years. A common feature of most of the series is that they extend right up to 1950. In some of the rivers regulation has been carried out, but it has not been considered that regulation has affected the annual volume of water. On the other hand, reconstituted water-discharge figures have been applied in a number of rivers in calculating the half-year figures for natural resources in hydraulic power. These particulars have in most cases been obtained from SMHI.

Some Data Characteristic for Sweden

The hydraulic power resources of a country are dependent on the quantity of water available and the differences in level. The water supply is determined by the precipitation and evaporation; it includes that part of the infiltrated water which reaches the surface again and the evaporation includes the water aspirated by the vegetation. The head of the water is related to the topographical conditions of the country.

In Sweden the topographical conditions are characterised by comparatively high mountain regions in the north-western part of the country, known as the Scandinavian mountain range. The lakes in that region are situated at an altitude of 500-700 m. above sea level. East and west of the mountain range stretches the north Swedish inland, which consists of an extensive primary-rock plateau sloping gently to the south-east, in which large lakes are also located. In this region, 300-500 m. high, the rivers have cut out deep channels on their way to sea level at the Gulf of Bothnia.

The configuration in northern Sweden gradually changes towards the south into the central lowland, which extends right across the country between the Baltic and the Aland Seas in the east to Skagerack in the west. In the middle of the remaining part of the country lies the south Swedish highland, which slopes down towards the coast in east and west and in the south to the south takes the form of lowlands consisting of a primary-rock plain carrying many lakes.

TABLE I. GROSS MEAN HYDRO-ELECTRIC POTENTIAL IN SWEDEN

	Natural energy resources	Proportion during winter half-year	Resources worth developing	Do. in relation to natural energy resources
	TWh/yr.	%	TWh/yr.	%
Upper Norrland				
Lule älv	32·2	20·5	14·0	43·5
Pite älv	8·5	22·3	3·6	42·4
Skellefte älv	7·8	27·0	4·0	51·3
Ume älv	23·4	23·9	10·7	45·7
Others, ex. Torne älv	17·9	22·4	6·2	34·7
Total	89·8	22·5	38·5	42·9
Cen. and Lower Norrland				
Ångermanälven	21·9	25·1	11·0	50·2
Indalsälven	21·6	25·6	11·0	51·0
Ljungan	5·8	27·6	2·5	43·1
Ljusnan	10·5	25·7	4·8	45·7
Others	1·2	41·7	0·2	16·7
Total	61·0	25·9	29·5	48·4
Rest of Sweden				
Dalecarlia and Mälar Provinces (Dalälven-Mälar)	15·4	33·1	5·5	35·7
South Sweden (Nyköpingsån-Lagan)	5·4	55·6	1·8	33·3
West Sweden	12·9	48·1	4·7	36·5
Total	33·7	42·4	12·0	35·6
Grand Total	184·5	27·3	80·0	43·4
Torne älv (Swedish part)	11·6	21·6		
WHOLE COUNTRY	196·1	27·0		

The precipitation in Sweden varies considerably and is most abundant in the mountain regions and on the west side of the southern highland region and this circumstance is particularly beneficial from the hydraulic power point of view. On average the precipitation in Sweden amounts to about 600 mm. per annum. The regions with the heaviest precipitation are situated at the source of Lule älv, Faxälven and Indalsälven, where the precipitation in some limited areas exceeds 1,200 mm. per annum. In the highland region in the south the precipitation amounts to 800-1,000 mm. per annum. On the eastern slopes of the mountains the precipitation is relatively low and the region with the smallest precipitation is in that part of Lapland bordering on Finland.

The precipitation is greater in summer than in the winter. The winter precipitation is largely in the form of snow, which means that the flow in the winter half-year (October-March) is insignificant, especially in the northern part of the country. Thus at the mouth of the Lule älv the flow in the winter half-year amounts on the average to 22% of the annual flow, whereas the corresponding percentage for Lagan in south-west Sweden amounts to 57%.

Evaporation has been calculated to average 250 mm. in Sweden. It varies considerably in different parts of the country and is very low in the mountain regions—only about 150 mm. In the southern parts of the country the evaporation is much greater and amounts to about 360 mm. Evaporation occurs, of course, mainly in the summer half-year, which explains why the flow in winter in the south Swedish rivers is greater than in summer, though the reverse applies to the precipitation.

The water discharge in the rivers is at present being taken continuously at about 300 places and, in addition, the water level is observed continuously at about 300 more places. Knowledge of water discharge is consequently very good.

Calculation Results

The results of the calculation of gross hydro-electric potential in Sweden are shown in Figs. 2 and 3, which are maps of isopleths, that is, curves which link together areas with the same energy supply per unit of surface. The isopleths in Fig. 2 have been drawn on the basis of calculations according to Method I, whereas Fig. 3 is based on Method II. It is evident from these maps that the major part of the annual hydraulic power supply is located in north Sweden, and this is supported by the maps showing the summer supply. On the other hand, the winter maps show that the energy supply per unit of surface varies but slightly during the winter half-year in the different parts of the country.

Table I gives an analysis of the final result. The total resources of natural energy in hydraulic power are thus estimated at 196·1 TWh per annum, of which something over 80% is situated in Norrland. These resources are distributed to the extent of only 27% in the winter half-year (October-March) so that 73% are available in the summer half-year (April-September). Table I also shows that the winter share of hydraulic power is appreciably smaller in the northern parts of the country than in the southern parts.

Sweden's total area is 449,200 sq. km. The gross hydro-electric potential per sq. km. is thus calculated at 0·44 GWh per annum per sq. km. on the average. The gross hydro-electric potential per sq. km. calculated according to Method I shows yearly values amounting to 3·53 GWh per sq. km., which refers to the lower part of the Ume älv, and the highest value for the summer half-year is 2·59 GWh per sq. km., which is to be found in the same surface section. Calculated according to Method II the corresponding highest values amount to 4·41 GWh per sq. km. for the whole year in the upper catchment area of the Lule älv and to 3·86 GWh per sq. km. for the summer half-year. The resources per sq. km. in the winter half-year are considerably less in north Sweden. On the other hand, in the southern parts of Sweden the values are much more uniform, as may be gathered from the accompanying maps.

There has also been included in Table I the amount of the economically exploitable hydraulic power. This is estimated to be 43% of the natural energy resources. It may be noted how this ranges from 40% to 50% in the big Norrland rivers to slightly more than 30% in the rivers in the southern part of Sweden.

In a calculation of this kind, the accuracy of the result will be the greater the more numerous and the smaller the surface sections into which the precipitation area of each river is divided. In the present case, the sections have at times been made comparatively large, for practical reasons. As the precipitation is greatest in the highest mountain areas, the result will be rather on the low side.

Though the hydrological material employed in the calculation may be regarded as very good, yet it is naturally subject to some error. Under these circumstances, it can be justifiably stated, in conclusion to what has been said, that the gross mean hydro-electric potential in Sweden is 200 TWh per annum in round figures.

Crimped-on Cable Glands. A publication received from the Combination Metallic Packing Co. (1921) Ltd., of Newcastle upon Tyne, refers to their crimped-on cable glands which have been the subject of thorough tests by Messrs. Merz & McLellan, the consulting engineers. The booklet is fully illustrated and descriptions are given of the various glands available.

Auto-Klean in Industry. Auto-Klean Strainers Limited have recently published a new brochure which describes their comprehensive range of filters and illustrates some of their more unusual applications in industrial engineering. This firm has a background of 38 years of experience and its aim is to assist manufacturers, designers and engineers to find the economic solution to their own individual filtration requirements.

Hydraulic Turbines. Under this title the Hydraulic Division of the Dominion Engineering Co. Ltd., Canada, have published a handsome brochure which gives a fully illustrated account of some of the work they have done and are doing in connection with turbines and valves for the hydro-electric industry. Some very large units have been produced, the Francis machines for the Bersimis II station being no less than 180,000 h.p. Also illustrated are the 150,000 h.p. impulse runners for the Kemano power house of the Aluminum Company of Canada. Other sections of the brochure deal with propeller turbines, Kaplan turbines, shut-off valves, research and development.

Uplift in Gravity Dams

In this series the author reviews the various conceptions of the phenomenon of uplift and presents his own analysis supported by experimental evidence. This article commences a discussion on the basic principles of the modern uplift theory

By SERGE LELIAVSKY, Ph.D., M.I.C.E., M.Am.Soc.C.E.

PART TWO

ALL the uplift theories described in the first article had one point in common, which was the metaphysical character of the assumptions from which they were derived. In contradistinction to this transcendental, abstract approach, the modern solution of the uplift problem tends to dispense with the conventional assumptions altogether, and considers seepage in dams as a natural phenomenon, capable of being observed and analysed in the same manner as all other physical processes belonging to the scope of experimental science.

Imagine a slice of unit thickness of a dam, as shown in Fig. 11. Seepage flow in the pores intersected by the plane *abcd* will occur as shown by the arrow. It is important to realise that, being extremely slow, this is essentially laminar flow, and therefore, the discharge is proportional to the gradient, and the hydraulic losses at the entrance, i.e., at *ab*, are negligible. It follows that the pressure diagram for the seepage flow along the plane *abcd* is represented by the prism *abcdef*. From the scientific standpoint no reduction of seepage pressure at the upstream face of the dam can be reasonably suggested or justified.

Let us now consider the area over which the interstitial pressure is effective. This, indeed, is the crux of the whole problem, which may, perhaps, be described as one of the most controversial questions in up-to-date technical literature. The existence of the pores is an uncontroverted fact, because every solid is capable of absorbing a certain amount of water, provided, that is, that the pressure is raised sufficiently high. Another fact, which is also a certainty, is that the entire area *abcd* cannot be occupied by the pores, for part of that area must, of necessity, be solid. Otherwise, the material could not be able to resist even a small amount of tension and shear.

But what is the ratio of the solid part to the total area is the main problem that the modern uplift theory attempts to solve. Since the ideal solids we are referring to are too small to be observed visually, we have to imagine other means for deriving the required information about the microstructure of these solids.

In this connection it must be understood that the difficulties of visual observation herein referred to do not belong to the class capable of being eventually solved by improving the purely technical details of the laboratory equipment, such, for instance, as increasing the magnifying power of a lens. In fact, the obstacle in the way of direct observation is that the characteristics of the architectural arrangements of the micro-solids in question are believed to be smaller than the mechanical parameters of the wave of visible light. In addition to which there is, also, the fact that the pore area over which the seepage pressure is effective, depends on the manner in which these micro-solids fail when fracture actually occurs. For instance, micro-failure by shear will produce a larger effective uplift area than micro-failure by tension. This, and various other questions bearing on the same problem, will be discussed more explicitly in a later article. For the present, in order to systematise the presentation of the relevant theory, we shall make an arbitrary assumption regarding the shape of the pores, viz., we shall suppose that the micro-structure of the material of the dam is as shown in Fig. 10*A*, and that *AB* in this drawing is the plane of rupture.

According to Delesse's law the ratio of the accumulated sectional area occupied by the voids to the total area of the section will then be equal to the volumetric porosity, which we shall denote by the symbol λ. Hence, with reference to Fig. 10,

$$\frac{\text{area } bc\text{—area } de\text{—area } fg}{\text{area } ah} = \lambda$$

On the first glance it may appear obvious that in order to calculate the total uplift force for the plane *abcd*, we must multiply the pressure intensities represented by the prism *abcdef* by the accumulated area of the pores, thus excluding the area of the solids, which are inaccessible to the penetration of the water.

Should this be indeed the case, it would seem that the reduction coefficient f which we must introduce in calculating the total uplift force, is equal to the porosity λ. Hence $f = \lambda$

This equation incorporates the simplest of all possible assumptions logically consistent with the modern ideas about seepage in dams. Though there is no name of an author specifically associated with it, it is nevertheless quite popular with some of the less-informed writers on the subject.

Unfortunately, it is false. As will be shown in detail, the uplift factor f is not related numerically to the porosity. To supply an elementary proof of this statement it will suffice to consider the second diagram in the same figure (Fig. 10*B*). It shows a micro-pore arrangement, as suggested, in 1934, by Prof. Terzaghi. It will be obvious from this diagram that should failure take place as shown by *ab*, the interstitial pore area subject to uplift pressure would be almost 100% of the total; which shows that the deci-

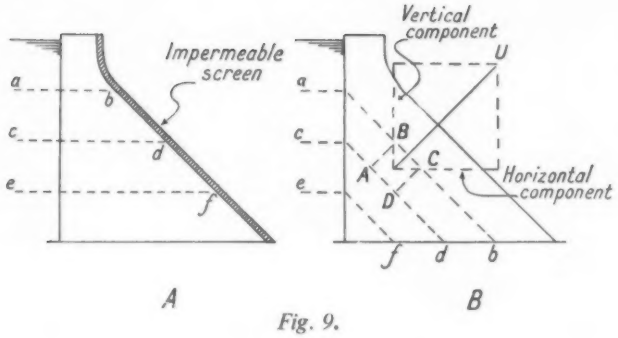


Fig. 9.

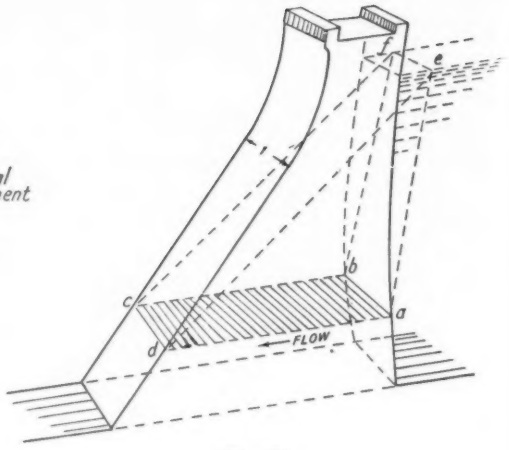


Fig. 11.

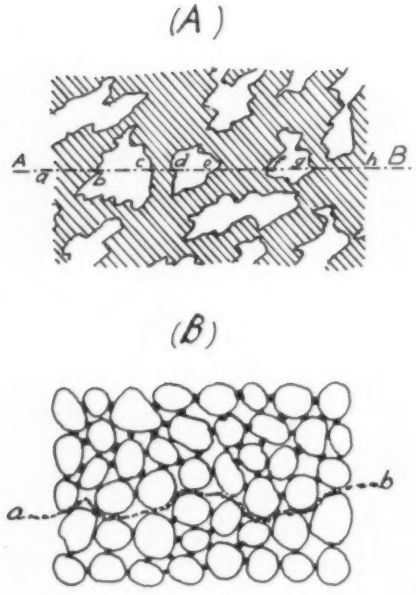


Fig. 10.

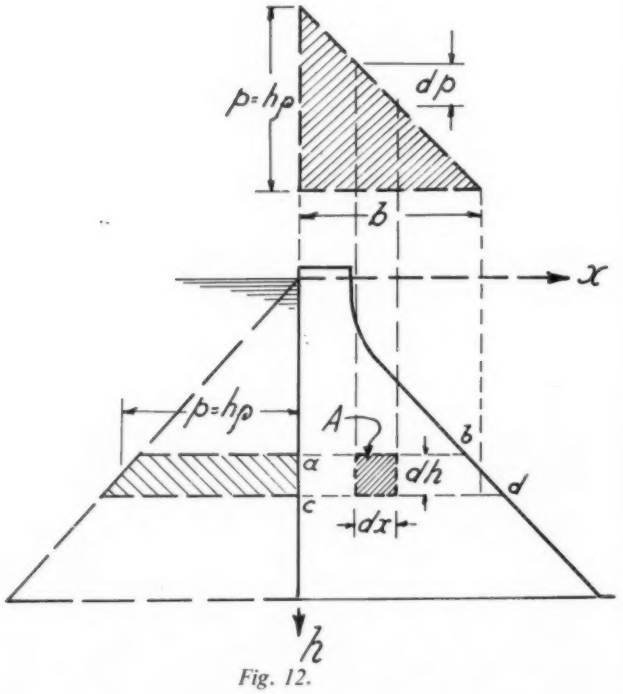


Fig. 12.

sive consideration in studying this problem is the arrangement of the pores and not their total area.

Fundamental Characteristics of the Uplift Force

Much of our present knowledge about the manner of action of the uplift force has been contributed by Prof. P. Fillunger and Prof. K. von Terzaghi. Some of Fillunger's ideas deserve to be quoted even in a short review of this theory. For instance, as early as 1913, he suggested the following approach to the problem in question. We imagine a porous dam with a watertight screen at its back, as shown in Fig. 9*A*. Since the screen will seal the exits from the pores, the water which has penetrated into the dam will remain at rest and the lines of equal pressure will therefore be horizontal. The uplift force, which represents the differences in these pressures, will, consequently, be vertical.

Suppose now, that the screen is suddenly removed (see Fig. 9*B*). Since water will start to escape from the pores, the pressures at the downstream face will begin to drop, until a state of dynamic equilibrium is arrived at, when the equal-pressure lines are parallel to the downstream face. In this state, the discharge filtering through the dam will be equal to the gradient multiplied by d'Arcy's coefficient.

Consider, then, the equilibrium of the prism *ABCD* (see figure). The pressures in the pores on the faces *AB* and *CD* are equal and opposite, and consequently balance each other; while the difference in the pressures applied respectively to *AD* and *BC* is a force—the force of uplift—which is normal to the

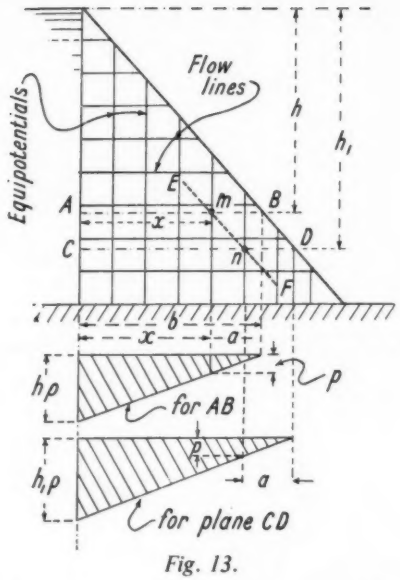


Fig. 13.

lines of equal pressures. It follows that the force which we call "uplift" is inclined and can, therefore, be resolved into vertical and horizontal components, as shown in the sketch.

Though this conclusion may appear paradoxical, there is no doubt that it is basically sound. The main question, however, is to assess correctly the interdependence between the so-called "horizontal" uplift, and the external horizontal water pressure applied to the face of the dam. In this connection it may be interesting, and even instructive, to recall a controversy which took place 15 years later, i.e., in 1928, in the pages of some French periodicals. It originated from the failure of the Habra dam, in Algeria, and began with a paper submitted to the French Academy by M. Baticle.

The author realised the existence of the horizontal uplift, which, at the time, was almost unknown, and by introducing this new force into the calculation, in addition to the external hydrostatic pressure, he concluded that, in order to be safe, the profile was to be widened in the proportion $\sqrt{2}$ to 1·0, as compared with Levy's section.

As might have been expected, the paper met with sharp opposition, for Levy's criterion was always believed to be over-conservative, and adding another 41% over and above this section seemed an extravagant waste.

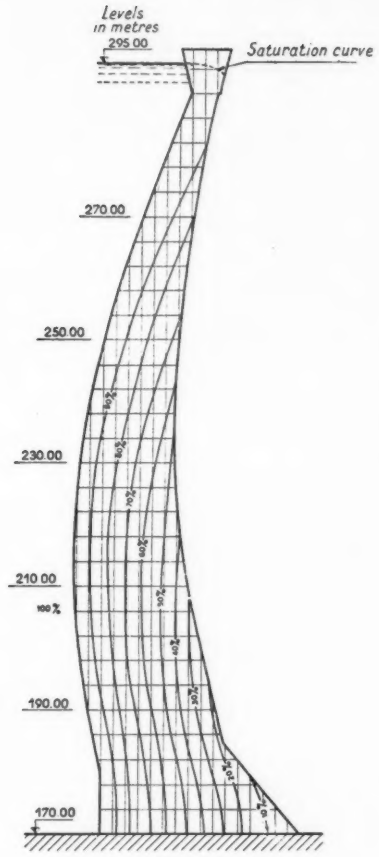


Fig. 17.

The problem was taken up in the press by another well-known French engineer, M. Pigeaud, who criticised Baticle's method of formulating his basic equations. To the credit of the latter it must be said that it did not take long to convince him to alter his views; in fact, Baticle's second paper, in which he admitted his error and modified his method, reached the Academy even before Pigeaud's objections were made public. But the most important contribution to this controversy was due to M. Jacques Menager, the same man who contributed so much to the development of prestressed concrete.

In the light of the ideas presented in this last article, the discussion on uplift which took place in French

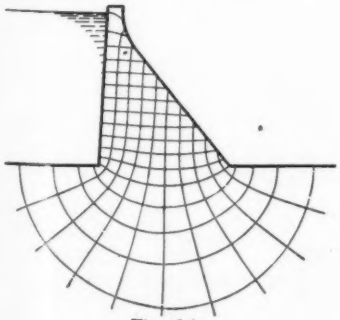


Fig. 14.

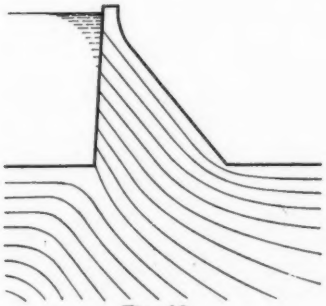


Fig. 15.

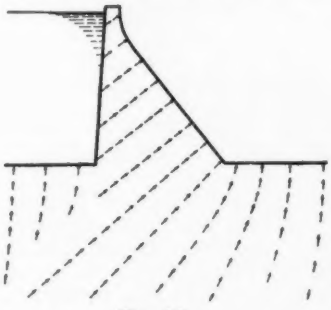


Fig. 16.

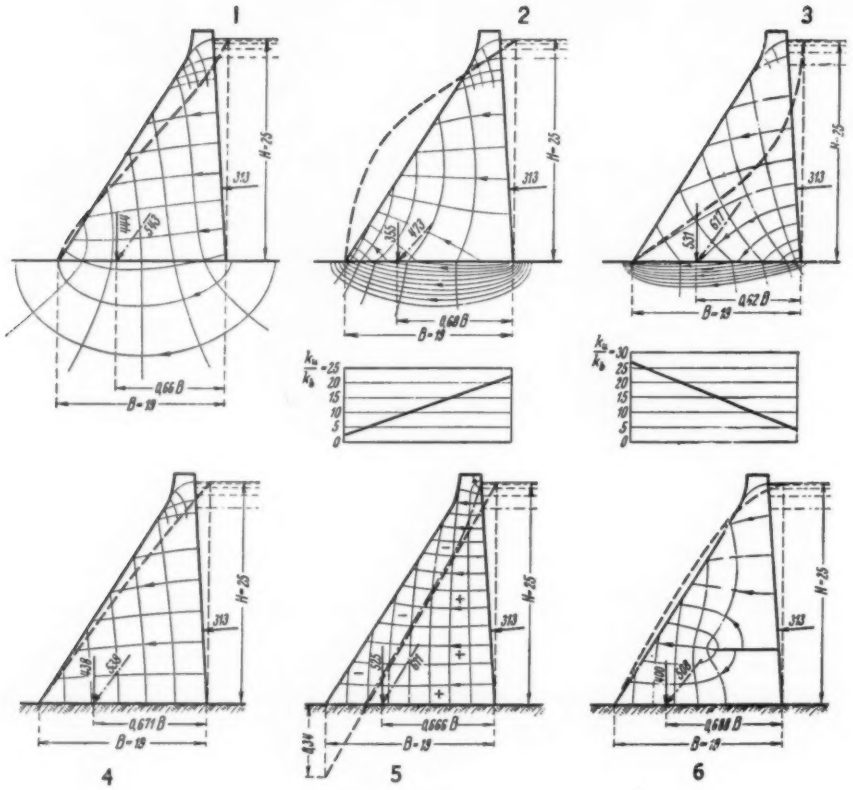


Fig. 18.

periodicals and the conclusions to which it led, may be presented as follows:—

(1) Consider two level planes *ab* and *cd* (see Fig 12) drawn at an infinitely small distance *dh* from one another. The pressure in the pores of the material enclosed between these planes varies from *p* at the water side to zero at the downstream face, as in the hatched diagram above the profile.

(2) It is fairly obvious that the horizontal load applied to the elementary parallelepiped *A* is equal to $dp \times dh$ (if we assume the coefficient *f* to be unity).

(3) Integrating all such loads, from $x=0$ to $x=b$, we get

$$\int_{x=0}^{x=b} dh \frac{dp}{dx} dx = pdh = h\rho \times dh$$

which is the force commonly assumed to be applied to the upstream face of the dam, and which gives

$$\int_{o}^{h} h\rho . dh = \frac{h^2\rho}{2}$$

if integrated for the full height *h*.

(4) This conclusion is essential for it shows that the force described as horizontal uplift is not a new item in the theory of stresses but is the well-known hydrostatic pressure, with the difference, however, that instead of acting on the face of the dam—as must have been the case had the latter been impermeable—it is now visualised as being diffused over the entire body of the porous structure.

(5) It will thus be obvious that the reason that Baticle obtained such extraordinary results in his

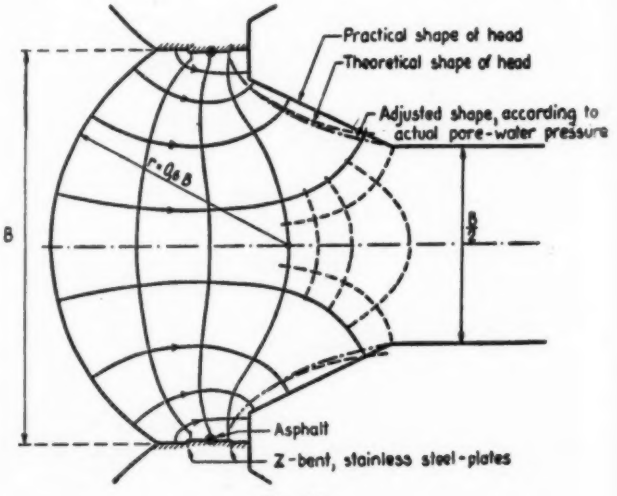


Fig. 19.

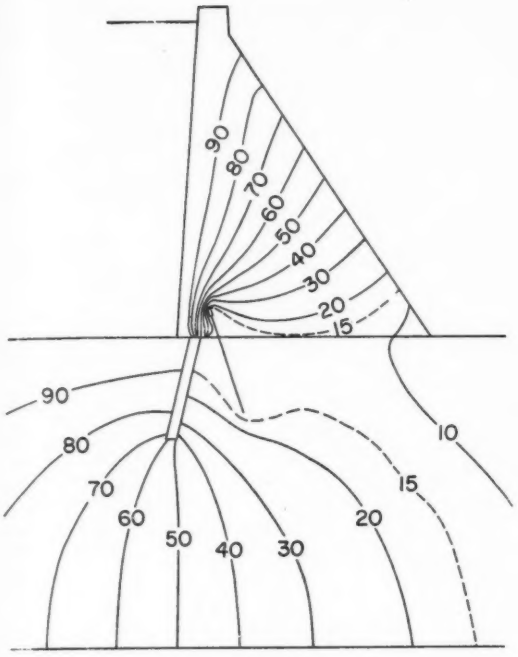


Fig. 20.

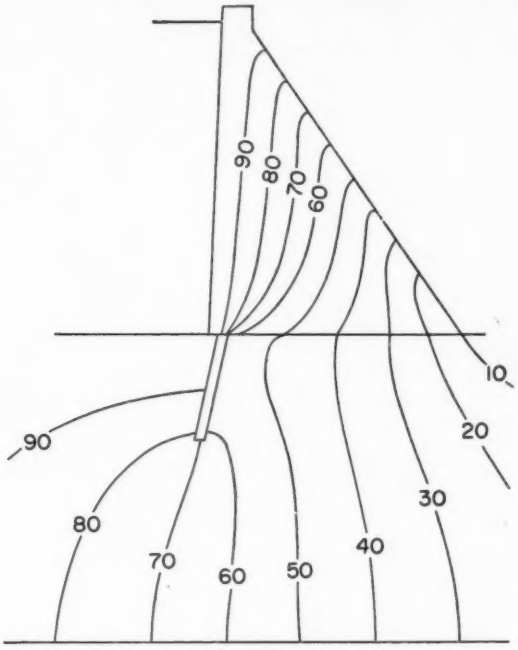


Fig. 21.

first paper was that he introduced in his calculations the same force twice—first as a hydraulic load applied to the face of the dam, and then again as an interstitial pressure distributed over the body of the structure.

Reference will now be made again to Fig. 12. Dividing the elementary force $dp \times dh$ by the volume $dh \times dx$ of the infinitesimal element to which it is applied, we obtain the intensity of the distributed interstitial pressure per unit volume, namely:—

$$\frac{dp \times dh}{dh \times dx} = \frac{dp}{dx} = \rho \frac{dh}{dx} = \rho s$$

in which h is the hydraulic head and s is the hydraulic gradient (for a horizontal section). We thus return to the same equation which was proposed by Terzaghi, in 1925, in his *Erdbaumechanik*, for the calculation of the stability of granular foundations beneath weirs and dams. In this manner, two different fields of engineering applications are united in one single theory—an achievement the significance of which cannot be overestimated. Note that in both cases we deal with "fields," which, as is well known, are best studied (either analytically or graphically) by the method of flow lines and equipotentials, i.e., flow-net diagrams.

Flow Nets for Dams

In investigating the problems relevant to the uplift pressure, we may either use the equipotentials, as is done by Terzaghi, or the lines of equal pressure, as was the case in the earlier computations of Fillunger. Both methods are equally correct, provided they are used in their proper ways.

Oscar Hoffman, in his book on "Seepage of Water and its Effect on Dams," published in Milan in 1928, appears to have been the first to have stated explicitly that for an elementary triangular dam profile (as in Fig. 13) on an impermeable foundation, the lines of flow are straight and horizontal, while the equipotentials (or lines of constant head) are also straight, but vertical.

Let us consider the conditions of the flow for any streamline AB (see Fig. 13). At its upstream end the pressure is $p = h\rho$ and at its downstream end it is zero. The pressure head expended in overcoming frictional resistance over the length $AB = b = h \tan \alpha$, is therefore h.

Hence, the gradient is equal to

$$s = \frac{h}{h \tan \alpha} = \cot \alpha$$

and consequently, the velocity is

$$V = \frac{k}{\tan \alpha}$$

where k is d'Arcy's coefficient.

Since this formula includes neither x nor y, it follows that the velocity is the same for any point of the field, which explains that on the Continent the ideal triangle is described as the profile of "uniform hydraulic resistance."

Consider now in the same figure (Fig. 13) any arbitrarily chosen point m, corresponding to the abscissa x. The pressure at this point is

$$p_1 = (h - sx)\,\rho = (b - x)\,s\rho = as\rho = \frac{\rho a}{\tan \alpha}$$

in which a is the horizontal distance from the downstream face. It follows that the lines of equal pressure are parallel to the downstream face. Curiously enough, we have already used this result, intuitively, in drawing Fig. 9.

The same author, Oscar Hoffman, shows in Fig. 14 the alterations required to Fig. 13 to convert the diagram to that of a practical dam profile. The basic rectangular network remains the same, but the

diagram comprises, also, the flow lines and equipotentials for the foundation, which, it will be observed, are generally similar to those beneath an open weir. In Figs. 15 and 16 the writer has drawn the equal-pressure lines and the loading system, based on Hoffman's original flow net.

On the other hand, in Fig. 18 Prof. Terzaghi has shown flow-net diagrams for dams under six different assumptions: (1) Similar to that of Hoffman; (2) and (3) in which the ratios of d'Arcy's coefficients for dam and foundation vary as shown in the diagrams below; (4) foundation impermeable; (5) effect of evaporation on downstream face and (6) effect of an open crack in the upstream face. All these diagrams illustrate conditions occurring in the usual types of gravity dams. In Fig. 17, which is due to Mr. Laginha Serafim, we have an example of a flow net for an arch dam, while in Fig. 19 Mr. Erling Reinius has studied the case of a mushroom head for a buttress dam.

The foregoing examples illustrate computation work done in the office, but flow nets can also be obtained experimentally, in the laboratory, by means of the well-known electrical-analogue method, first suggested by Pavlovsky. For instance, Figs. 20 and 21 represent two sets of equipotential curves obtained in that manner by Mr. Horace A. Johnson.

In Fig. 20 the foundation of the dam is provided with an impermeable cut-off and a line of drainage wells behind it; whereas in Fig. 21 the wells are dispensed with and the cut-off remains in place alone. The object of the experiment was to bring out the influence exerted by the wells on the general pattern of the curves shown in the diagram.

There is one point which emerges from these examples of flow-net diagrams, namely, that whatever method is used for drawing them, a flow net may always be computed for every case met with in practice. It follows that we can obtain a set of equal-pressure lines for every problem of dam design under consideration.

(*To be continued*)

Firming-up Hydro-electric Systems

The basic concept of integrating a gas turbine plant with a hydro-electric system is economy. A relatively small amount of supplementary thermal energy can economically "firm-up" secondary hydro-electric power which would otherwise not be available for dependable energy.

The B.C. Electric's Port Mann gas turbine plant is a good example of the recent advent of major gas turbine installations into hydro-electric systems. Four 25 MW Brown Boveri units are being installed in the plant, making it the largest of its type in the world. Principally, the plant will be used for "firming-up" secondary hydro power acting as a stand-by at peak power demands. For such services, gas turbine installations possess a number of technical advantages over conventional steam turbine plants, the principal advantages being, lower capital costs, a cold start-up to full load period of less than 1/10th the time required by steam turbines and a positive power correction at lower ambient temperatures. The greater efficiency of a steam turbine does not have too great a bearing, as with a low load factor the operation costs are small compared to fixed costs.

The overall thermal efficiency of a gas turbine installation varies from 18 to 28%, depending on the cycle of the unit. Basically, a gas turbine cycle consists of a compressor and turbine on a single shaft with a combustion chamber in the air ducts between compressor and turbine. A careful study of the economics surrounding each installation will permit a decision as to whether equipment should be added at increased capital cost to improve the thermal efficiency. Such a study has led to the Port Mann gas turbine installations to be of a two-shaft, two-stage, intercooled, re-heat, open cycle nature. At full load, overall thermal efficiency will be in the order of 22%. In case of complete electrical system failure, the turbine units can be started by auxiliary stand-by diesel electric generators. The plant installation will cost, exclusive of the switchyard, approximately $12 million or $120 per kW. Fuel for the turbines will principally be natural gas with crude oil as an alternative. Approximate daily fuel consumption for each unit will be 11 million cu. ft. of natural gas or 48,000 gallons of crude oil.

To operate this plant manually would necessitate a staff at the site at all times, thus increasing costs by 50%. It is principally for this reason that a complete remote and supervisory control of the plant is being installed, the cost of which is estimated to be $2·50 per kW. Recording instruments are used to document critical operation data and also to permit an analysis of shut downs caused by abnormal operating conditions. In addition, the plant is completely self-protected and is independent of the supervisory operator when operated on local automatic.

A.E.G. Progress. The beautifully illustrated issue No. 4 for 1958 of the official house organ for Allgemeine Elektricitäts-Gesellschaft (A.E.G.) contains no less than thirteen articles on a variety of subjects of interest to engineers concerned with high-voltage transmission. The publication is entitled "A.E.G. Progress" and it contains, *inter alia*, articles on: high-voltage switchgear; the switching of large currents; methods for testing the switching capacity of high-voltage circuit breakers; high-voltage switch cubicles; support insulators, bushings and isolated switches for high-voltage switchgear; and isolating switches for very high voltages.

New Welding Process. An effective demonstration was given recently of the "Comet" welding process at the works of Rockweld Limited, Grafton Road, Croydon. In this process the arc is struck between a composite continuous wire and the workpiece, the welding taking place in an atmosphere of carbon dioxide. The composite wire contains both the deoxidisation agent and the slag-forming materials, and the carbon dioxide protects the metal from atmospheric contamination in the weld. It is thus a visible arc process and can readily be adapted to both automatic and semi-automatic operation. The carbon dioxide used is normal cylinder gas, fed through a preheater to a pressure regulator and flowmeter, and is introduced around the arc by means of an annular nozzle.

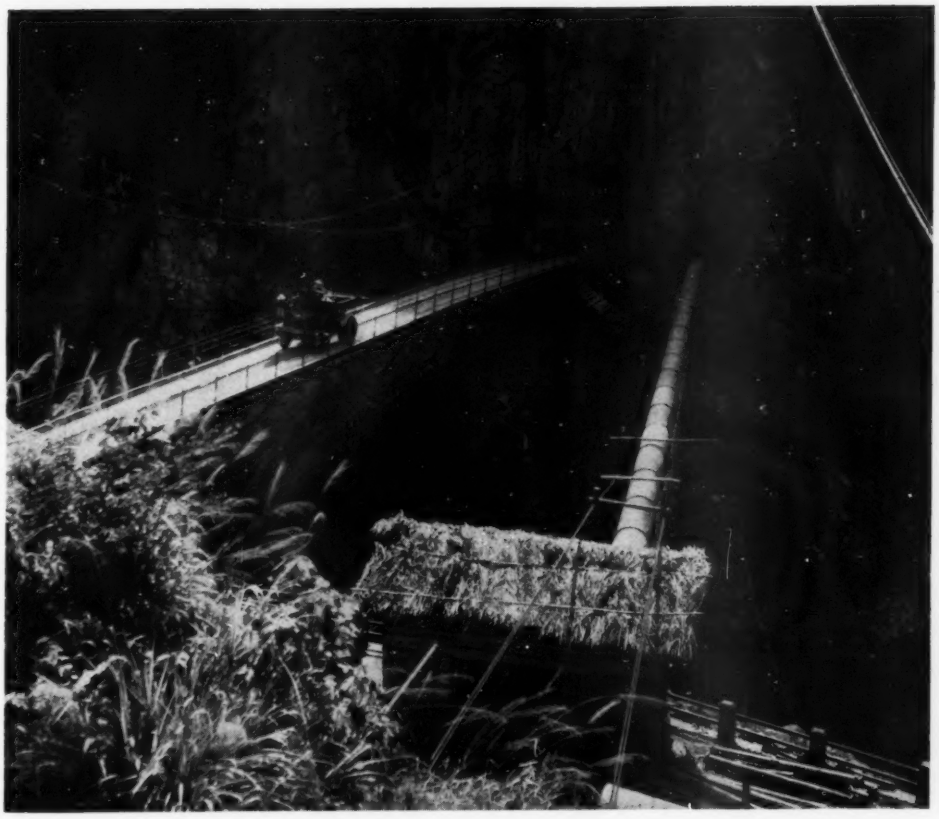


Fig. 1. Temporary suspension bridge leading to Ku-Kuan station

Hydro-Electric Power in Formosa

A survey of the Ta-Chia valley development scheme consisting of a vast hydro-electric power project undertaken by Nationalist China and which will take ten years to complete.

THE ever-increasing demand for power in Formosa (Taiwan), situated 100 miles east of Fukien province on the Chinese mainland, has made it necessary for the Nationalist Chinese Government to embark on an extensive scheme of power production to absorb the expanding demand. Early exploration of the rocky and mountainous interior revealed the Ta-Chia valley as a major source of hydro-electric potential, and with the construction of the all-important east-west highway shown in Fig. 4 in the vicinity of this valley, the commencement of this large scheme became feasible. This road, which was cut through virtually inaccessible and difficult terrain, runs parallel to the Ta-Chia river on which the hydro-electric scheme is centred and made possible the exploration and development of the new natural resources abundant there, not only hydro-electric but also of mineral wealth and large timber areas.

The Ta-Chia valley is located in the north-western part of the island and is surrounded by mountain peaks that reach a height of 13,000 ft. while the river, of the same name, winds its way through solid rock gorges.

Present Power Resources

The present power output in Formosa totals 1,100 MW and this is obtained from both hydro and thermal power stations. This represents a tremendous gain from the time when the Taiwan Power Company took over a damaged chain of stations operated by the Japanese and embarked upon a programme of rebuilding and power-plant expansion.

Recently commissioned, the latest hydro-electric project adding to the national grid is the Lungchien scheme which is situated near Hualien on the eastern coast of the island. It consists of a power station built underground and cost U.S. $10,717,000. The Lung-chien station is linked downstream on the Mu-kua

Fig. 3. The construction road, rope railway and Ma-hw offi

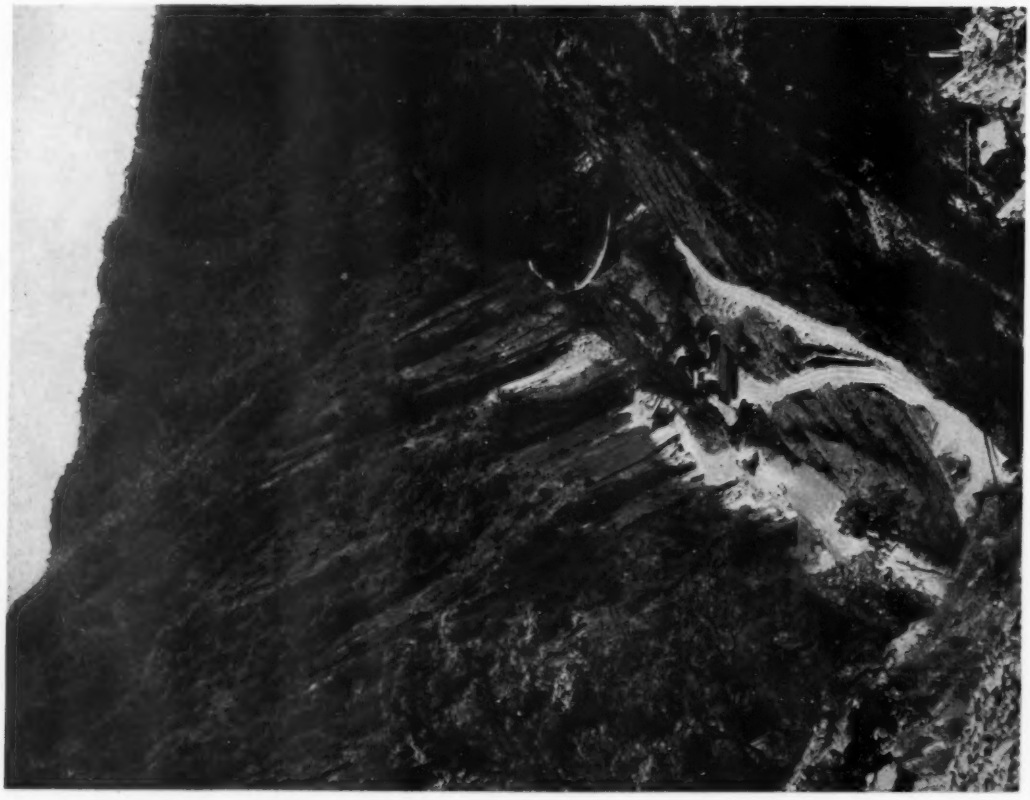


Fig. 2. General view of the working site

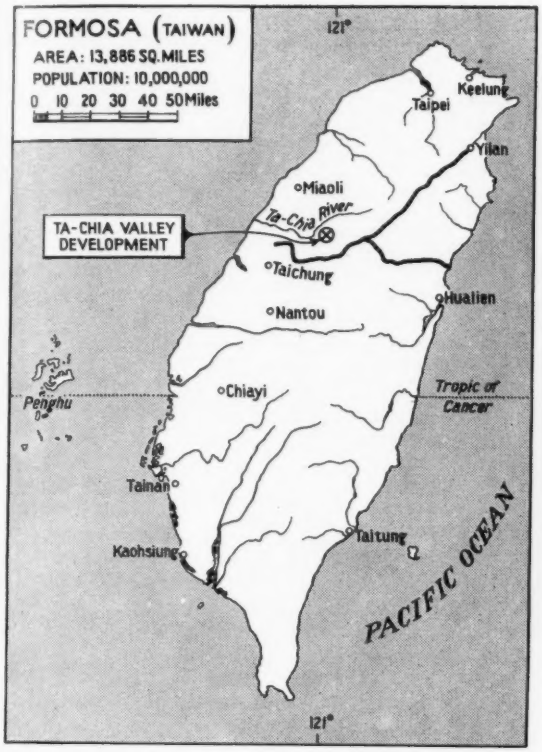


Fig. 4. Location map of Ta-Chia scheme

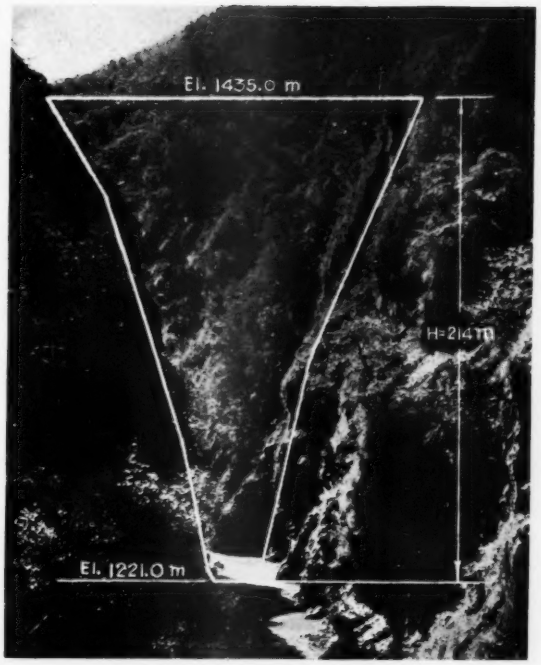


Fig. 5. A view of the dam site

river with the Tungmen hydro-electric power plant which was completed in 1955 and which is also underground. To the south of Ta-Chia valley is Sun Moon Lake, one of the most celebrated scenic spots in Formosa and near it is the Wu-sheh hydro project, with a capacity of 40 MW and linked with a second power station of like capacity. American funds totalling $10 million were provided for its completion.

Financial Aid

To carry out many of the water-power development schemes already commissioned and others projected for future expansion, the Nationalist Chinese Government has obtained substantial financial help from the United States. This aid is not on a give-away basis but is to be repaid with interest into a counterpart fund supported by both the United States and Nationalist Chinese Governments and so become available for other development projects. It is, in effect, a banking service earning money on its investments and re-used many times for such purposes. In addition, the Nationalist Chinese Government has committed funds of its own from power earnings which have appreciably increased with the extension of its facilities in recent years.

The Ta-Chia valley development scheme is estimated to cost $158 million, half of which will come from Chinese funds and the balance from American aid in dollar grants or counterpart currency.

Proposed Development

The Ta-Chia valley scheme, on which construction has already begun, consists of an elaborate hydro-electric and water-supply system which, when completed, will perhaps make the island the most self-sufficient power-producing nation in the Far East. It is expected that all the installations planned will be in operation by 1970 when Formosa's combined electric power system will reach the capacity of 2,400 MW firm power, of which 1,300 MW will be produced by the Ta-Chia scheme.

The Ta-Chia development involves the construction of six power stations and two concrete dams one of which, the Ta-chien dam, will be the dominant construction feature of the entire project. This dam, situated at the eastern end of Ta-Chia valley at an elevation of 5,000 ft., is expected to be one of the highest arch-type concrete dams in the world. Its present design calls for it to reach a height of 800 ft., nearly 80 ft. higher than Hoover dam in Boulder Canyon in the U.S.A. However, the Hoover dam design permits expansion to a proposed height of 922 ft. and when it was finished and operable in 1936, had cost $160 million for power capacity of 1,249 MW. For comparison, the Ta-Chia valley's projected costs will be in the region of $158 million and will have a capacity of 1,300 MW. To judge the size of this dam it is interesting to note that the Mauvoisin dam, now under construction in Switzerland, will be 780 ft. high when finished, while India's Bhakra dam measures a hundred feet less. It will require 750,000 cu. m. of concrete and will create a reservoir in the hills holding 500 million cu. m. of water.

The second dam is of a more modest size and will act as an auxiliary to the Ta-chien dam. This is the Ku-kuan dam which will be 300 ft. in height and will need 72,720 cu. m. of concrete for its construction. It will then form a regulating pond of only 8·6 million cu. m. of water and will augment the flow to the power

stations when necessary. The reservoirs formed by the two dams will have enough impounded water to supply the six power stations in this development scheme and also be sufficient to irrigate an area of 53,000 acres of farmland in the surrounding country.

Power Stations

The six power stations incorporated in this development scheme are as follow:—

Tien-lun station	80 MW	in operation
Ku-kuan station	180 MW	being built
Ma-an station	70 MW	in design
Shi-kang station	100 MW	in design
Chin-shan station	480 MW	in design
Ta-chien station	420 MW	in design

Construction of the power plants in the Ta-Chia valley has been under way for some time, and in fact, one of the power stations is already in operation. The Tien-lun plant, located at the western perimeter of the valley uses the present flow of the Ta-Chia river for its hydro power. This same river flow will be utilised for the Ku-kuan station which will eventually be linked to the other stations in the valley. Work has recently begun on this station which is to be of the underground type to render it immune to floods, landslides, earth tremors, and the typhoons which are common in that area. The construction progress is very satisfactory on the Ku-kuan station and it was 30% complete by mid 1959. The station will have a capacity of 180 MW and will contain four generating sets, two of which will be ready by 1961, the second pair to be installed three years later. A labour force of nearly 1,300 men is engaged on the Ku-kuan project alone and the construction also involves cutting access roads through rock gorges. These have retaining walls against landslides and involve the building of five bridges to facilitate the movement of men and supplies. The suspension bridge shown in Fig. 1 can sustain a load of 30 tons and crosses the gorge hundreds of feet above the river level. It leads directly into the tunnel entrance of the power station. Down in the valley is the Ma-lun adit. This is a construction tunnel giving access to Ku-kuan's pressure tunnel. It also opens the way to extending the excavating operations at several locations. A rope haulage at a 45° incline connects the construction road at the top of the valley with the working areas at the lower levels.

At present, a construction crew of 1,200 Chinese engineers and workers is engaged in boring tunnels through the hills to divert the course of the Ta-Chia river which is being harnessed for the project while work soon begins on the Ku-kuan dam itself. Construction has also begun on initial phases of the pressure tunnel which will draw regulated water from Ku-kuan reservoir to the Tien-lun and Ku-kuan power stations.

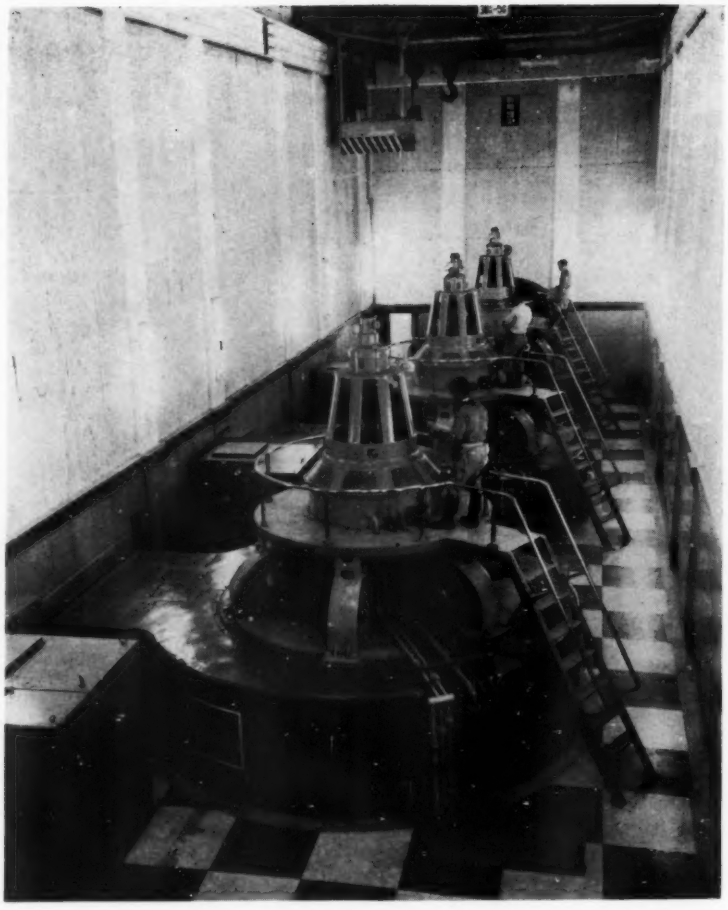


Fig. 6. Underground turbine hall at Tungmen

Once these projects are finished, other phases of the Ta-Chia valley development will be launched, the next major task being the construction of Ta-chien dam. It is anticipated that this will be ready by 1965 and designs for the remaining power stations, now being drafted, are expected to be completed some three years afterwards.

As electric power demands in Formosa have been increasing at more than 10% annually over the last decade—an effect generally attributed to the effects of the Second World War—the Ta-Chia scheme will, when complete, be sufficient for the island's needs for 20 years or more and will be able to absorb the growing demand.

It should be noted that all the major work in the planning and construction of this large project is being conducted by Chinese engineers and surveyors who are graduates of local universities, though there is a reliance on an American consulting engineering firm which has two resident engineers in the Ta-Chia valley. There are two French resident engineers as well.

Acknowledgments

We are indebted for the supply of information and photographs of this scheme to Hamilton Wright Organization Inc., of 30 Rockefeller Plaza, New York, U.S.A.

Fig. 1. Puntledge generating plant is in the left-hand building

Some Hydraulic Features of Puntledge Generating Plant*

Puntledge generating plant has an exceptionally long penstock in relation to head, and pressure fluctuations are limited by means of an automatic pressure regulator. The automatic and remote-control features of the plant are described in relation to this equipment. In this first article the reasons for the design adopted are outlined, and the testing and control features are described

By A. W. LASH† and R. E. PASSMORE§

PART ONE

PUNTLEDGE Generating Plant of the British Columbia Power Commission is a fully automatic plant, with remote supervisory control. The plant has an unusually long penstock in relation to the developed head. Limitation of pressure surges in this penstock for lower-cost penstock design, therefore, was an important consideration in the economy of the project.

General Description of Plant

The power development at Puntledge replaces an older and smaller one in the same location, constructed by Canadian Colleries (Dunsmuir) Ltd., in 1912. The same diversion dam was used for the new development, but all other parts of the plant were rebuilt. The new plant, which feeds into the Commission's 138 kV Vancouver Island transmission system, was commissioned in 1955.

Puntledge plant consists essentially of a low diversion dam on the Puntledge river which directs the water into a penstock approximately three miles long, leading to a single generating unit. The penstock varies from 12 ft. in diameter at the intake to 9 ft. at the power house and consists of approximately 4,500 ft. of woodstave pipe and 12,500 ft. of steel pipe.

The steel portion of the penstock is of all-welded construction with expansion joints averaging 1,000 ft. apart, the maximum distance between joints being 1,600 ft. Concrete anchor blocks are provided at horizontal and vertical bends and at changes in pipe diameter, the expansion joints being placed midway between anchor blocks. The pipe is supported by bedding in the ground to within 1 ft. of its horizontal diameter, the embedded portion being protected against external corrosion by a coating consisting of two layers of coal-tar pitch-base paint, followed

* This article forms the substance of a paper, but with additional material, presented to the December 1958 Annual General Meeting of the American Society of Mechanical Engineers.

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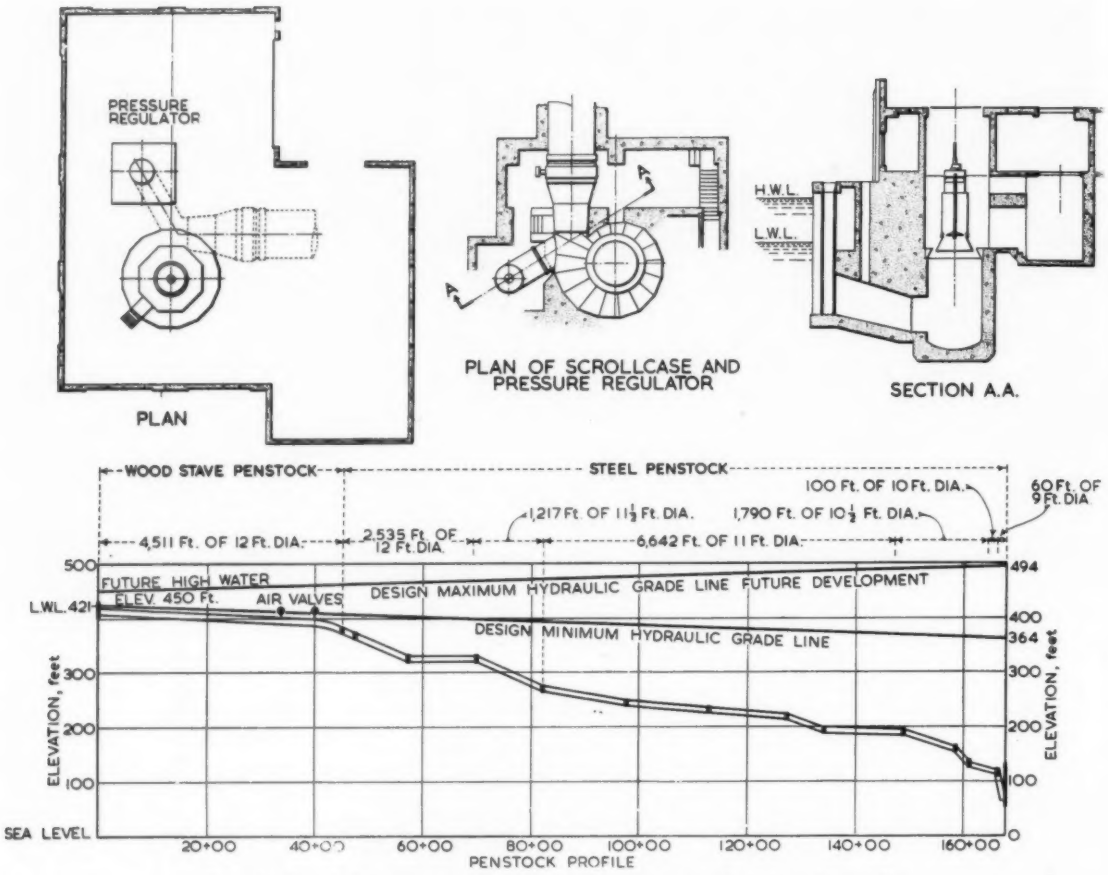

Fig. 2. Puntledge power house—general layout and penstock profile

by a layer of asbestos fabric and finished with a final coat of paint.

The turbine is of the vertical-shaft Francis type. It is rated 35,000 h.p. at 340 ft. net head and drives a 30,000 kVA, 0·9 p.f. generator, the speed being 277 r.p.m.

The hydraulic equipment at the power house includes a horizontally pivoted butterfly valve 108 in. in diameter and a pressure regulator connected to the turbine scrollcase. This discharges excess water rejected by the turbine through an energy-absorption chamber discharging into the river below tailwater level. The governor is of the cabinet-actuator type.

The turbine, butterfly valve, and pressure regulator were supplied by Canadian Allis-Chalmers Ltd., the generator by Canadian Westinghouse Ltd., and the hydraulic governor by Woodward Governor Company.

The general layout of the power house, as indicated in Fig. 2 is fairly conventional, and as the plant is normally unattended and is in no sense a show place, the design was made on strictly utilitarian lines.

Operation of the plant is normally automatic with supervisory remote control from the Commission's John Hart generating station, some 40 miles distant. A resident attendant is retained on call at the site, and if necessary, the plant can be operated manually.

Basis of Design

Puntledge plant is intended generally to operate on base load, generating the maximum possible energy from the available water supply. The principal reasons leading to the decision to provide this type of plant were as follows:—

1. The exceptionally long conduit in relation to head. Due to the physical characteristics of the site this length could not economically be reduced.

2. Increasing the diameter of the conduit to suit larger installed generating capacity would have been costly in relation to increased capacity obtained.

3. The closeness of the hydraulic gradient to the top of the woodstave pipe required some means of limiting rapid increases in demand for water which might cause negative pressure conditions within the pipe. For this reason the plant could not lend itself to dealing with rapidly fluctuating loads.

4. The interconnected system contains other plants where adequate peak-power facilities can be added more economically than at Puntledge.

5. The pond upstream of the dam is too small to provide for greatly fluctuating demands. It had been hoped to remedy this in part by increasing the height of the dam, but this has not as yet been found desirable due to possible interference with a fish-spawning area.

6. Due to fishery requirements it is necessary to

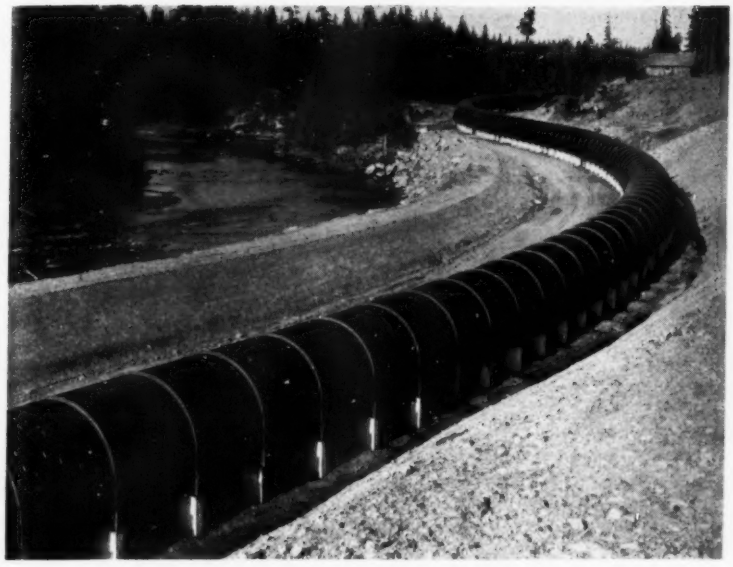

Fig. 3. Part of the woodstave penstock

Fig. 4. Part of the 12 ft. diameter steel penstock

provide a continuous and sizeable flow of water in the river dowstream of the power house, even if the plant is shut down. A bypass valve at the powerhouse was necessary to meet this requirement.

While Puntledge plant is normally operated in parallel with other plants of the Vancouver Island system, it may, in emergencies causing interruption to power supply from these plants, be required to operate isolated. In such circumstances flow would be established through the penstock and the relief valve would function as a synchronous bypass, on a "water-wasting" basis.

Elimination of Surge Tank

To provide regulation in a base-load plant for the conditions as outlined, two schemes were possible—either an appropriate pressure regulator, or a surge tank primarily to meet load-off conditions, with an opening-rate-limiting device on the governor.

Due to the comparatively uniform slope of the conduit, a surge tank located near the power house would have been of exceptional size. For instance, an appropriate location for a surge tank would be in the vicinity of Station 160+00, Fig 2, but a tank located here, if of the differential type, would be about 350 ft. high and have a diameter of about 28 ft. The cost, including foundations, was estimated at about $600,000.

It was found that a pressure-regulating valve, costing about $100,000 installed, would perform the same duty as the surge tank, of meeting requirement 6, previously mentioned, of maintaining adequate river flow in all circumstances. It was, therefore, decided to eliminate all surge-tank provision and to provide complete regulation by this means.

Design and Operation of Pressure-regulating Valve

Basically the pressure regulator is required to perform two functions: (*a*) to establish or shut off flow in the penstock; (*b*) to open and close automatically as a synchronous bypass in conformity with turbine-gate movements.

The specifications called for the pressure regulator to be designed to discharge full turbine flow and to limit the pressure surges in the penstock to within 20% of the net turbine head.

Normally the plant runs with the pressure regulator closed. By interlocking arrangements, to be described in the second article, the turbine gates cannot be opened farther than the limiting position set by the regulator. Should the gates close, the regulator opens and discharges water rejected by the turbine, then it may be slowly closed to any required extent.

The butterfly-type shutoff valve is located just upstream from the turbine. This valve, when closed, performs the usual functions of preventing loss of water from leakage through the turbine gates and consequent deterioration of gates and adjacent surfaces, and allows the turbine to be inspected and repaired without draining the pipeline. It can also be used as an emergency-shutdown device and is designed to close under full conduit flow in event

that the turbine gates or the pressure regulator cannot be closed.

Operation of the turbine gates and governor and also of the pressure regulator and shutoff valve, is by oil pressure from a single system. This system operates at a pressure of 300 lb. per sq. in. and includes two 40 h.p. pumps with a large pressure tank. The pressure tank with one pump operating has sufficient capacity to operate the pressure regulator, shutoff valve, and turbine servo-motors, simultaneously. The second pump is held in reserve.

Water-hammer Calculations

The conduit was designed so that normal stresses in the steel would not exceed 15,000 lb. per sq. in. for a pressure rise of 20% also that in no circumstances would the hydraulic grade line fall below the top of the pipe. Water-hammer calculations were made with the objective of determining suitable rates for opening and closing the shutoff valve, regulator, and wicket gates to satisfy these requirements. Arithmetical integration methods as set forth in George R. Rich's book, "Hydraulic Transients," were used in dealing with this problem.

Before commencing water-hammer calculations it is necessary to establish with as close a degree as possible the acoustic velocity of pressure waves and the pipe friction.

Acoustic Velocity. The velocity of elastic wave travel in a thin-walled steel pipe can be calculated by

$$a = \frac{4675}{1 + \frac{KD}{Ese}}$$

where a = wave velocity, ft. per sec.
K = modulus of elasticity of water, 294,000 lb. per sq. in.
D = internal pipe diameter, in.
E_s = modulus of elasticity of steel, 29,400,000 lb. per sq. in.
e = thickness of steel plate, in.

This formula does not make any allowance for the method of support of the pipeline previously described, and some doubt existed regarding the effect on acoustic-wave velocity of the pipe bedding and of the concrete encasement at the anchor blocks.

The wave velocity in the woodstave section was computed by the same method as for the steel pipe, allowance being made for the composite characteristics of the pipe material, the modified formula being

$$a = \frac{4675}{1 + \frac{KD}{E_w b + E_s \omega}}$$

where a, K, D, and E_s are as previously noted
E_w = modulus of elasticity of wood, 1,600,000 lb. per sq. in.
b = thickness of woodstave, in.
ω = area of steel bands per inch-length of pipe.

The same precision of results of computations of wave velocity cannot be expected in the case of woodstave pipes as for steel.

The computed weighted-average wave velocity for the whole pipeline worked out at 2,422 ft. per sec. Test results gave 2,435 ft. per sec. It seems likely, therefore, that the conventional formula can be applied with close accuracy to this type of pipeline, the method of support adopted in this case having little influence on the wave velocity.

On the basis of an acoustic-wave velocity of 2,422 ft. per sec., the reflection time of the conduit (16,905 ft. long) is 13·96 sec.

In computing conduit pressure surges, an extreme possible range of acoustic-wave velocity of 2,000 to 3,000 ft. per sec. was considered.

Friction Factor. The conduit friction factor was computed by using Hazen and William's coefficients on the basis of two extreme conditions:

Worst 30-yr-old steel pipe, H. W. coefficient, 90
30-yr-old woodstave pipe, H. W. coefficient, 110
Best new steel pipe, H. W. coefficient, 120
new, woodstave pipe, H. W. coefficient, 145

The resulting factor C in the formula $F = CV^2$ worked out as 0·512 for worst and 0·298 for new condition. For the new condition this gave a friction loss with the pressure regulator full open of 32·42 ft. Tests gave 31·1 ft.

Based on the assumed pipeline characteristics, and the requirements as to pressure variation, it was found that the complete opening and closing rate of the pressure-regulator stroke should not be faster than 6 minutes.

Speed Rise and Runaway Conditions

The specified generator-inertia effect was 4,100,000 lb. ft². However, the manufacturer had developed a generator having an inertia effect of 5,400,000 lb. ft²., and on the basis of this machine being installed the turbine manufacturer guaranteed a

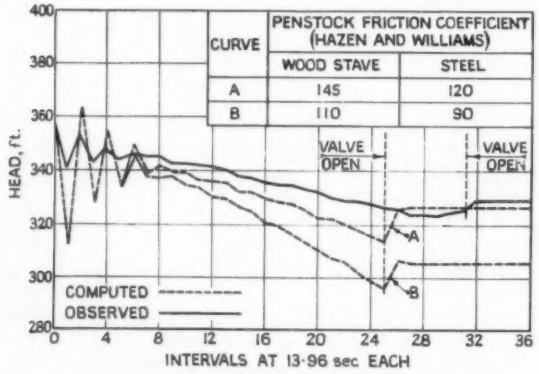

Fig. 5 Computed and observed penstock pressure variations, no load to full load

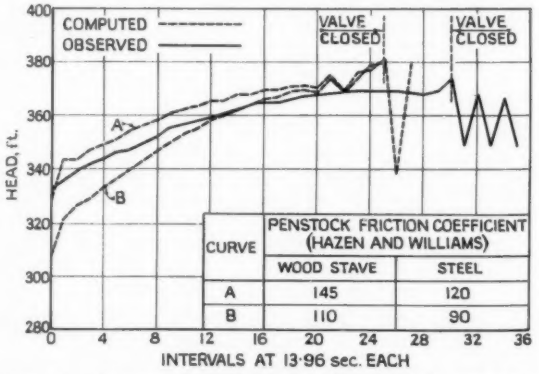

Fig. 6. Computed and observed penstock pressure variations, full load to no load

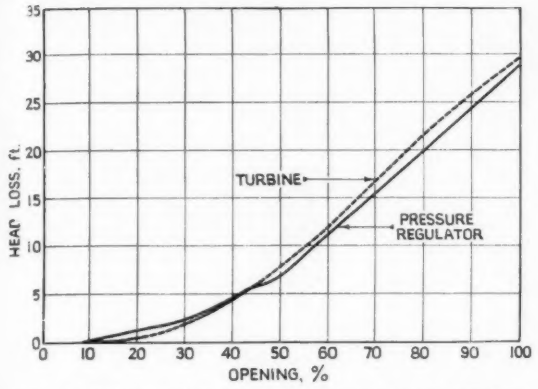

Fig. 7. Relation between head loss and gate opening

maximum speed rise for total-load rejection of 20% based on 4 sec. complete governor stroke (an effective time of 3 sec. with 1 sec. allowed for cushioning at the end of the servo-motor stroke). Tests disclosed on actual speed rise of 21%, which was considered sufficiently close to the guarantee.

For a generating unit of this specific speed (35·5), the speed rise with the wicket gates jammed full open and with full load rejection, should not exceed a maximum value of 180% of the rated speed, and should level off to a stable speed of about 175% of rated speed.

The maximum computed pressure rise under runaway conditions was 27·5%. While somewhat higher than the pressure rise specified under normal operating conditions, in view of the low probability of such a runaway occurring, this computed pressure rise was considered acceptable.

Tests and Adjustments

Reference has already been made to the results of some of the tests made before the plant was accepted for operation. Briefly the main tests carried out were as follows:

1. On the Pressure Regulator. A series of tests was made to determine maximum pressure variation in the pipeline due to varying times for opening and closing the pressure regulator. This speed is controlled by a small d.c. motor driving through appropriate gearing.

The finally adjusted periods for complete opening and closing of the pressure regulator were established at 7 min. 24 sec. and 7 min. 2 sec., respectively—or a little slower than was indicated as safe by the computations.

A comparison of computed and observed pressure variations during opening and closing is given in Figs 5 and 6.

2. On the Turbine. The main tests on the turbine included:

(*a*) *Load Tests.* These aimed at establishing the turbine-gate-opening/output relationship.

(*b*) *Throttle-Valve Setting.* A throttle valve on the oil-supply line is provided to prevent the turbine gates from closing rapidly on load rejection should the pressure regulator fail to open. This valve was adjusted so as to close the turbine gates in 9 min. 20 sec. for the full stroke.

(*c*) *Wicket-Gate Timing.* The turbine-gate timing is 32 sec. for full-opening stroke and 4 sec. for a closing stroke. The slow opening speed is a compromise found best adapted to meet three requirements:—

(1) Limitation of pressure variations arising from operating time lag between regulator and gate movements, and slight differences between rates of discharge at certain proportional openings. As previously noted, the upper part of the penstock is close to the hydraulic gradient, and is, therefore, particularly vulnerable to negative pressure surges.

(2) Slow gate opening gives a measure of safeguard to the penstock in event that the regulator should fail to close for any reason as the turbine gates open.

(3) This speed has been found sufficient for "get-away" purposes when starting up. No damage to the generator thrust bearing, which is of the Kingsbury type, occurs at this speed unless the unit has been shut down for a considerable period of time. In such event the generator should, before starting, be raised by hand pump—a manual operation.

The fast closing rate of 4 sec. was found to be satisfactory with reference to penstock pressure rise arising from time delay in operation of the pressure regulator and with regard to turbine overspeed on load rejection.

(*d*) *Speed-rise Test.* It was considered undesirable

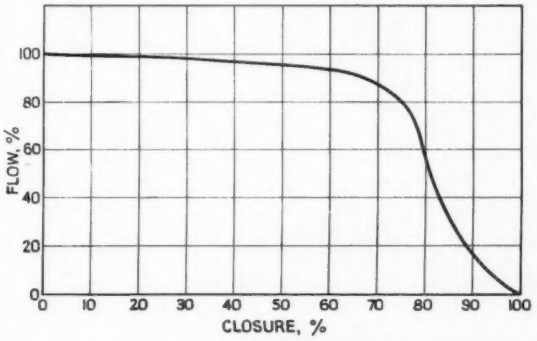

Fig. 8. Derived curve for butterfly valve

and unnecessary to make tests of the speed rise with the turbine gates jammed full open and with full load rejection; therefore, none were conducted. The only speed-rise tests were those to determine the speed rise on allowing the gates to close normally from full open in a governor period of 4 sec. Reference has previously been made to the result of this test. The overspeed shut down was set at 360 r.p.m.

3. On the Pressure-Regulator/Turbine Correlation. This group of tests was the most important one conducted, the object being to ensure correct correlation between wicket gate and pressure-regulator movements to limit pressure variations to within the specified requirements.

At the outset, as was to be expected, wide divergence between the turbine and regulator-discharge curves was found to exist. The total possible turbine discharge was found to be considerably in excess of the regulator discharge. However, the maximum regulator discharge was sufficient to give more than the specified

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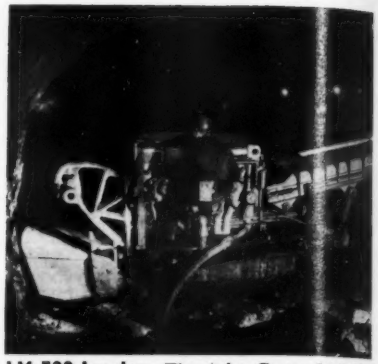


LM 500 Loader. The Atlas Copco LM 500 is a high-capacity, crawler-mounted shovel loader with conveyor belt discharge. Used for large-scale mining and tunnelling projects, the LM 500 greatly speeds up the mucking operation. Variable discharge height permits the use of most types of dumpers and the smooth conveyor discharge also allows truck transport.

The LM 500 has a wide clean-up range: a 3-metre wide area can be cleaned up without turning the crawler tracks. In addition, the LM 500 is the only crawler-mounted loader with swivelling top deck—a feature that saves time and considerably reduces the crawler track wear.

Rail-Mounted Loaders. Atlas Copco make a series of rail-mounted loaders with bucket capacities as shown below:

LM 30	0.14 m³	(equiv. in cu. ft.)	5 cu. ft.
LM 56	0.20 „	„	7 „
LM 100	0.30 „	„	11 „
LM 200	0.70 „	„	25 „

Designed for maximum reliability, these high-capacity loaders incorporate the latest features to reduce operator fatigue and increase safety.

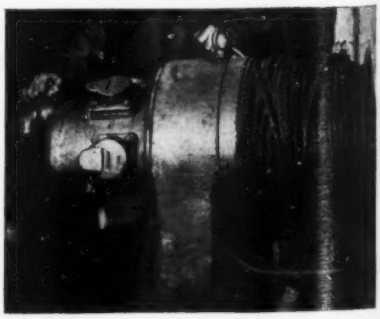
T2G Auto-Loader. With only one man required to control it, T2G Auto-Loader carries out the complete mucking operation—loading, transporting, dumping. Fitted with pneumatic tyres, its great manoeuvrability makes for excellent performances in wide areas as well as confined spaces.

Used throughout the mining industry, this Auto-Loader has been proved to increase efficiency to a startling degree.

Other Atlas Copco Auto-Loaders available are the T2GH and the T4G.

U3N Shuttle Dumper. This sturdy machine makes an ideal partner for the Atlas Copco loaders. The tramming engines of both shuttle dumper and loader can be synchronised to work in tandem, adding to the shovel impact and substantially increasing the net loader capacity. Strongly built and easily operated, the U3N shuttle dumper has a long range—750 feet (230 m) using ¾" hose, and a loading capacity of 70 cubic feet (2m³).

Six-cylinder Winches. These Atlas Copco winches give smoother hauling from creep to full speed and a powerful starting torque. The MHK82, shown here, is the largest model. There are other Atlas Copco winches in several sizes to meet varying requirements.



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turbine output of 35,000 h.p. at 340 ft. net head. In other words, the turbine was found to be more oversize than the regulator. To meet this condition the extreme turbine-gate opening was limited and blocked at 82%. Following the making of this provision, detailed adjustment of the control lines, cables, and cams of the pressure regulator was made to bring about approximate coincidence of the two discharge curves. Fig 7 shows the relation between head loss and gate opening as finally established for the turbine and regulator.

4. On the Butterfly Valve. In event of failure of the pressure regulator to close, it might be found desirable to close the butterfly valve under full-flow conditions. As expected, tests on the butterfly valve showed that only about 10% of the discharge is cut off during the first two-thirds of the stroke, the remaining 90% being cut off during the last third. Fig. 8 is a derived curve for the butterfly valve obtained by comparing a pressure-recorder graph of valve closure with a similar curve for the pressure regulator. In view of these conditions, the full closure stroke of the butterfly valve was established at 21 min. 7 sec. As the butterfly valve was specified to open only under balanced-pressure conditions, the full opening time was set at 7 min. 20 sec.

Tests were made of closing under varying flow conditions, including full-open pressure regulator. The valve closed satisfactorily, although with some noise and vibration from the 15th to the 20th min. of closure.

Control and Operation

The principle followed in operating this plant is to establish steady penstock pressure before initiating any control action. The unit can then be brought into operation in several different ways, but the one usually adopted is as follows:—

1. After checking that the shutoff valve is open, the pressure regulator is opened usually to about 70% of capacity and is left open to this extent for at least 1 min. to allow any pressure surges in the pipeline to die down before the turbine is started.

2. The generator can then be started. The turbine gates will normally open about 30% while the generator is coming up to speed before dropping back to about 5% when the generator is at normal speed.

3. The pressure regulator is then adjusted to the load position required.

4. The final stage is to raise the governor speed control to a point beyond the fully loaded position. The turbine then takes over the water supply from the pressure regulator until this is fully closed. It cannot take more load than that corresponding to the load-position setting of the pressure regulator.

Oil-Pressure System. The turbine, pressure regulator, and butterfly valve are operated by a common oil system of sufficient capacity to operate them simultaneously.

Two oil pumps, each with a capacity large enough to handle normal loss of pressure are provided, the installation being so arranged that if one pump cannot maintain system pressure, the other will cut in.

Control of the hydraulic components of the plant should oil pressure fail is different from that usually adopted in hydro-electric power stations where a loss of oil pressure initiates a shutdown sequence which closes first the turbine gates and then the shutoff valve. At Puntledge, due to the slow rate of closing of the shutoff valve and, in addition, the pressure regulator, this could not be done. If this normal procedure were followed, it would be possible to lose oil pressure before the turbine, pressure regulator, and shutoff valve could be closed. The shutoff valve and the pressure regulator would then open fully and the turbine gates drift open to about 20% in a very short period. Since this could cause damage to the penstock, a different procedure has been adopted as follows:—

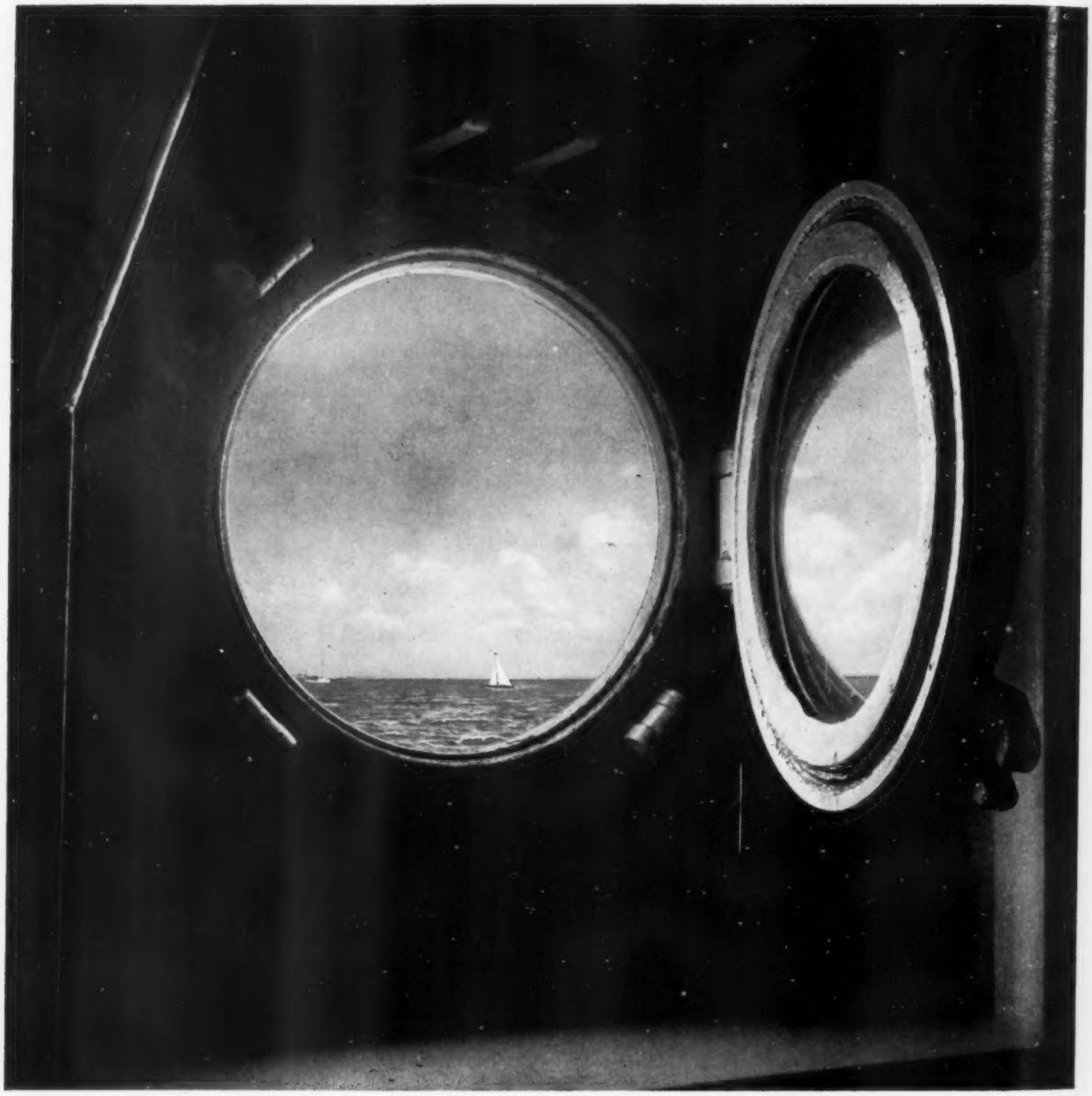
If oil pressure falls to 260 lb. per sq. in. an alarm will ring. On further fall of pressure to 220 lb. per sq. in. an electrical contact is broken and the turbine gates close, the circuit breaker remains closed and the generator motors. Once the turbine gates have shut and the regulator has opened to its equivalent position, the control motor will run the regulator full open at its normal rate. The regulator thus opens initially in synchronism with the gates and finally at its controlled rate which prevents its opening under conditions which might damage the penstock. The shutoff valve will be left in its open position. As the oil pressure continues to fail, the turbine gates will drift open to their balance point of about 20%.

No other movement can then occur until oil pressure is restored. The pressure regulator cannot close a corresponding amount, but the gate-opening rate in this situation will prevent a large penstock pressure drop.

Automatic Control. Three methods of control are provided, manual, automatic and supervisory. Hydraulic considerations made it impracticable to include the shutoff valve and pressure regulator in the automatic sequence. Before starting, as already noted, the shutoff valve must be fully open and the pressure regulator should be opened about 70%.

The governor is equipped with a shutdown solenoid and a speed-no-load solenoid. Because of the slow gate-opening time it was found that for satisfactory or fast "breakaway" the gates had to be opened to approximately 30%. A special circuit was designed using an auxiliary relay and a gate limit switch to permit starting in this manner and still retain the usefulness of the speed-no-load solenoid. In receiving a start signal the control checks that conditions are correct for starting and then energises the master relay. The master relay picks up the shutdown solenoid and the speed-no-load solenoid to open the gates past the speed-no-load position and start the unit. When the gates reach 30% the gate-position switch and auxiliary relay de-energise the speed-no-load solenoid to bring the gates back quickly to just above the speed-no-load position. At approximately 50% voltage, the speed matching and auto-synchronising relays are energised to synchronise the unit on to the busbars. The magnetic-amplifier-type voltage regulator is applied when the generator breaker closes an auxiliary contact of which closes to energise the speed-no-load solenoid and permit the opening of the gates past the setting of the solenoid. The field rheostat is automatically preset before starting to ensure normal voltage on the unit and voltage matching is not required.

The relay protection on the generator is more or less standard and includes split phase, differential, current-balance, voltage-restrained over-current, over-



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voltage, loss of field, generator earth, field earth, and thermal overload relays.

SUPERVISORY CONTROL. The supervisory control points used directly for control of the generator are as follows:—

1. Supervise station-service breaker, telemeter 13·8 kV bus voltage.
2. Supervise generator breaker, telemeter generator megavars.
3. Start, stop, generator; supervise master relay.
4. Supervise generator lock-out relay.
5. Raise-lower voltage, telemeter generator voltage.
6. Raise-lower load, telemeter gate position.
7. Close-open pressure regulator, telemeter pressure-regulator limit position.
8. Supervise pressure regulator "not-closed fully-closed" position, telemeter generator frequency.
9. Supervise shutoff valve open-closed position.
10. Low battery-voltage alarm.
11. Generator creep alarm.
12. Station general alarm.

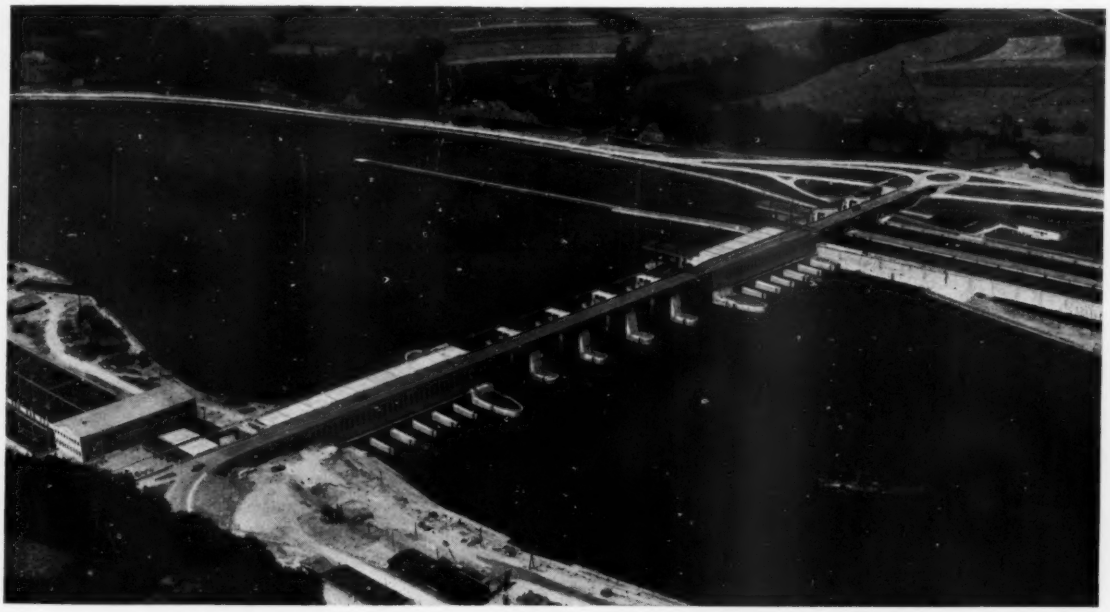
In addition, generator megawatts are telemetered continuously to John Hart station.

The supervisory control equipment is all-relay type, using a number of impulse codes with automatic check back for point selection. Two types of telemetering equipment are used, one using rate of impulse and the other length of impulse.

The carrier-current equipment for supervisory control is a broad-band type suitable also for carrier relaying. Narrow-band frequency-shift carrier is used for continuous and selective telemetering. A frequency-modulated, manual simplex communication channel provides communication between Puntledge and John Hart stations.

(To be continued)

No Opening Ceremony For Ybbs-Persenbeug



An aerial view of the Ybbs-Persenbeug hydro-electric scheme across the Danube

Ybbs-Persenbeug, one of the largest hydro-electric schemes in Europe, has now been completed and has all six generators in full production. Since the first two generating units were successfully installed in October 1957, nearly 1,800 million kWh of generated power have been delivered to the Austrian grid. During this summer the fully completed power station accomplished a mean daily output of nearly 3 million kWh, while the highest daily output of a single generating unit of 195·6 MW capacity was 4,650,600 kWh. During the month of July this year Ybbs-Persenbeug alone supplied almost 10% of the Austrian electricity demand and accounted for 31% of the Austrian export of electricity. This large hydro-electric scheme is expected to produce 1,274 million kWh in a normal year. The plant was fully described in WATER POWER June 1956, p. 297.

In spite of the significance of this scheme the committee of the Ybbs-Persenbeug plant have proposed to the directors of Österreichische Donaukraftwerke A.G. that there should be no official opening ceremony. This may appear puzzling in regard to such an outstanding achievement but the committee suggest that the anticipated expense of such a ceremony could be more beneficially donated to the fund established for the flood victims. Österreichische Donaukraftwerke A.G., however, believes that an official opening ceremony would give worthy emphasis to the inauguration of this plant and it is still possible that one may be held.

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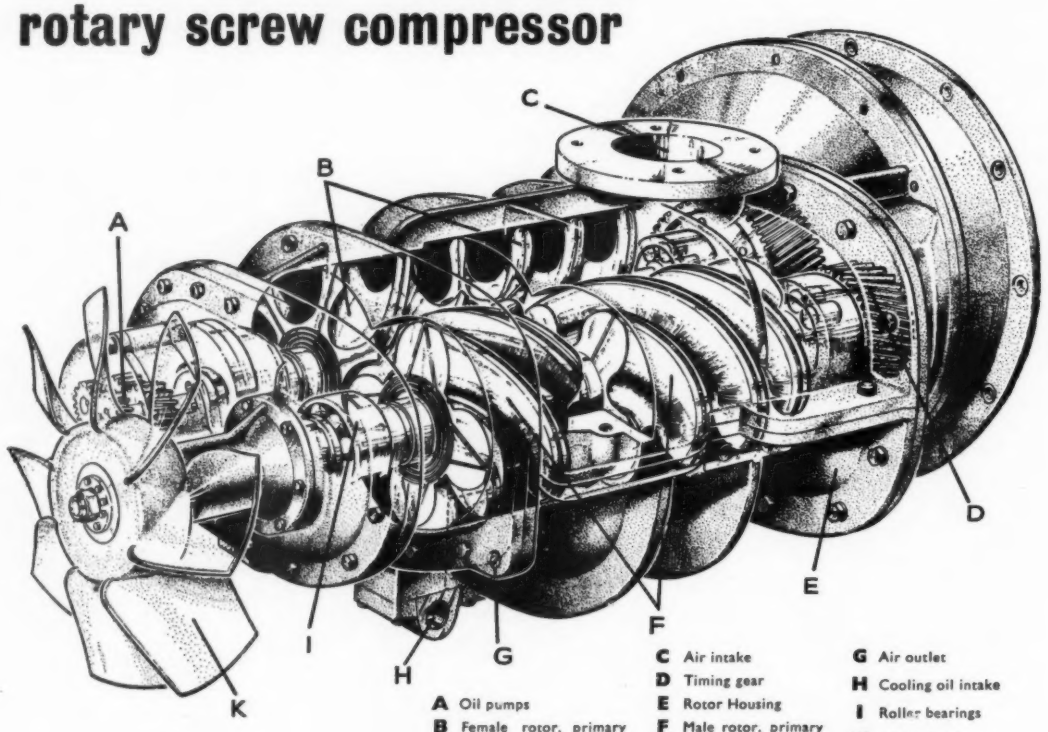

A Oil pumps
B Female rotor, primary and secondary stage
C Air intake
D Timing gear
E Rotor Housing
F Male rotor, primary and secondary stage
G Air outlet
H Cooling oil intake
I Roller bearings
K Oil-cooler fan

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Loch Awe Road Diversion

The inauguration of the Loch Awe project at the Pass of Brander recently was carried out by Mrs. A. A. Fulton, wife of the general manager of the North of Scotland Hydro-Electric Board. The crawler tractor used in the ceremony to cut the first sods for a road diversion consisted of a British-built International BTD-20 fitted with a Rolls-Royce engine. This particular tractor was carrying out work for A. M. Carmichael Limited, one of the largest civil engineering contractors in Scotland, the road diversion being carried out in advance of other work on the Loch Awe project to maintain the flow of main road traffic to Oban.

The Loch Awe pumped-storage project of the North of Scotland Hydro-Electric Board is due to be completed at a cost of approximately £26½ million, and will make use of the water power over an area of over 300 sq. miles. Electricity provided by the Hunterson atomic power station will pump water to a storage dam situated 1,000 ft. up Ben Cruachan. The power used to pump the water will be generated at night or at week-ends by nuclear means.

Mr. Fulton stated that the natural conditions so favoured the scheme that power would be produced more cheaply than by any other means, and added that the scheme would constitute one of the most advanced pumped storage plants in the world.

Protecting Sluice Gates at Beauharnois

The Beauharnois hydro-electric station is designed as an integral part of the power scheme developed jointly by the Hydro-Electric Power Commission of Ontario and the Power Authority of the State of New York, under the aegis of the International St. Lawrence River Board of Control. As with most projects of this type, the planning engineers were faced with the problem of ensuring adequate protection of the metal structures in contact with the water stream. This was made more difficult by the local conditions which include a wide seasonal variation in temperature, and a considerable quantity of mineral debris flowing from the International Rapids section of the river.

The coating system was required to give complete protection against corrosion for as long a period as possible, to maintain adhesion under widely varying conditions of temperature, to fit easily into the strict time schedule and, finally, to be economical in cost and application.

The systems suggested can be divided into four types:—

(*a*) conventional, multi-coat paint treatments, incorporating anti-corrosive primer, undercoat and finishing coats; (*b*) bituminous coatings; (*c*) single-coat preservative treatments of the non-oxidising type; (*d*) multi-coat application of synthetic, high polymer materials.

In fact, the choice made was of a (*c*) type material, the others being considered to be too costly to apply, having insufficient adhesion under all temperature conditions, difficulty of application, creep, etc. In consequence a non-oxidising material was stipulated, this consisting of "Camrex," manufactured by Camrex Paints Limited, Sunderland, England. This material can be applied by brush direct on to the prepared steel and, within 24 hours, is claimed to give the characteristic thick plastic coat, possessing extremely good adhesion. Moreover, the work done in this manner is immediately ready for service conditions.

Examination of the gates after a year's service show that the coating is behaving extremely well; no physical damage could be detected and the film still possessed all its original properties.

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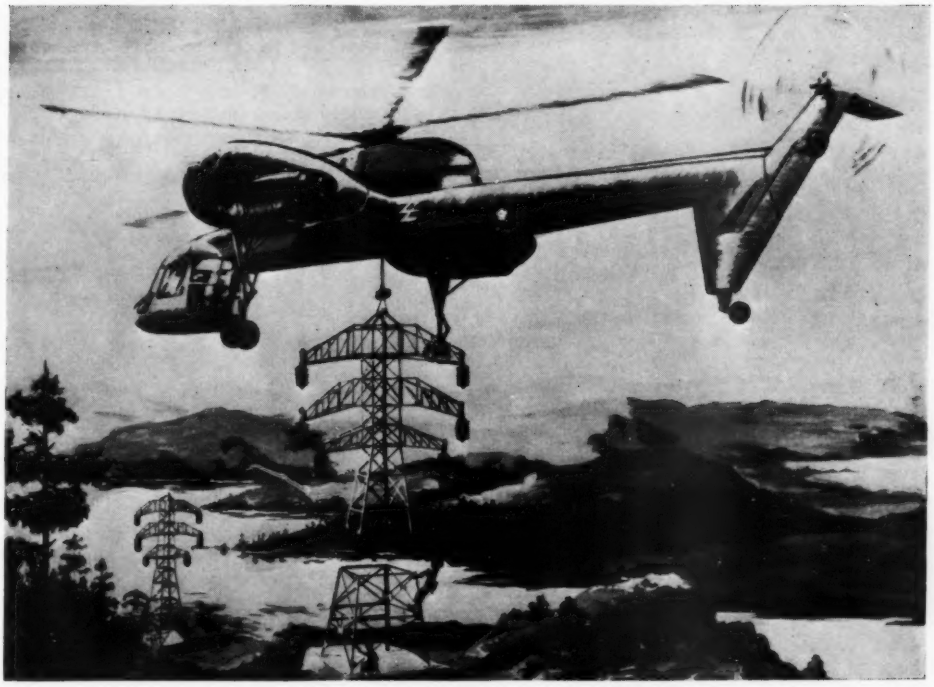

The Sikorsky 5-60 crane helicopter was demonstrated publicly for the first time early this year. The power units are 2,100 h.p. Pratt & Whitney engines which enable this load-carrying helicopter to lift from four to six tons over distances of up to 100 miles.

Loads are suspended from cables beneath the fuselage, which basically consists of a box-shaped boom section with a cockpit at one end and a tail rotor at the other. The unusual features of design include a fully rotating pilot's seat and an extra set of controls to enable the pilot to face toward the tail of the aircraft during the loading or unloading of cargo.

In the accompanying illustration, the Sikorsky 5-60 is shown moving the entire upper section of a transmission tower into position over mountainous and inaccessible territory. As previously discussed in this journal (see WATER POWER, April 1959, page 121), this is just one of the many ways in which helicopters can be used on hydro-electric schemes. Future developments in this type of aircraft should eventually make it possible to lift loads of up to 50 tons, and it is, therefore, reasonable to suppose that it will then be possible to transport, by air, practically all items of equipment and machinery required for hydro-electric projects.

Multi-purpose Michigan

The East Sussex River Board has chosen a tractor shovel made by Michigan (Gt. Britain) Ltd. to tackle the variety of work spread over the eastern half of England. The task is the maintenance of over 200 miles of river banks and estuary foreshore which present a diversity of mechanical handling problems. The Michigan 75A is very mobile, moving quickly from one area of operations to another over difficult terrain and proved efficient in soft ground due to the high flotation rubber-tyred wheels. It can have different attachments depending on the duty required, such as a 1¼ cu. yd. bucket, backfill blade and fork-lift. Since the maintenance work extends beyond the river banks and much land clearance has to be carried out, the Michigan is used to fill pot-holes, level the ground and carry away the spoil to a convenient point, thus no lorries or dumpers were necessary to be on the site, thus reducing costs. In the tidal estuary where the Board wages a continuous battle against shifting sands and silt, the machine works during the few restricted hours at low tide to gather silt and shingle and restore the river banks.

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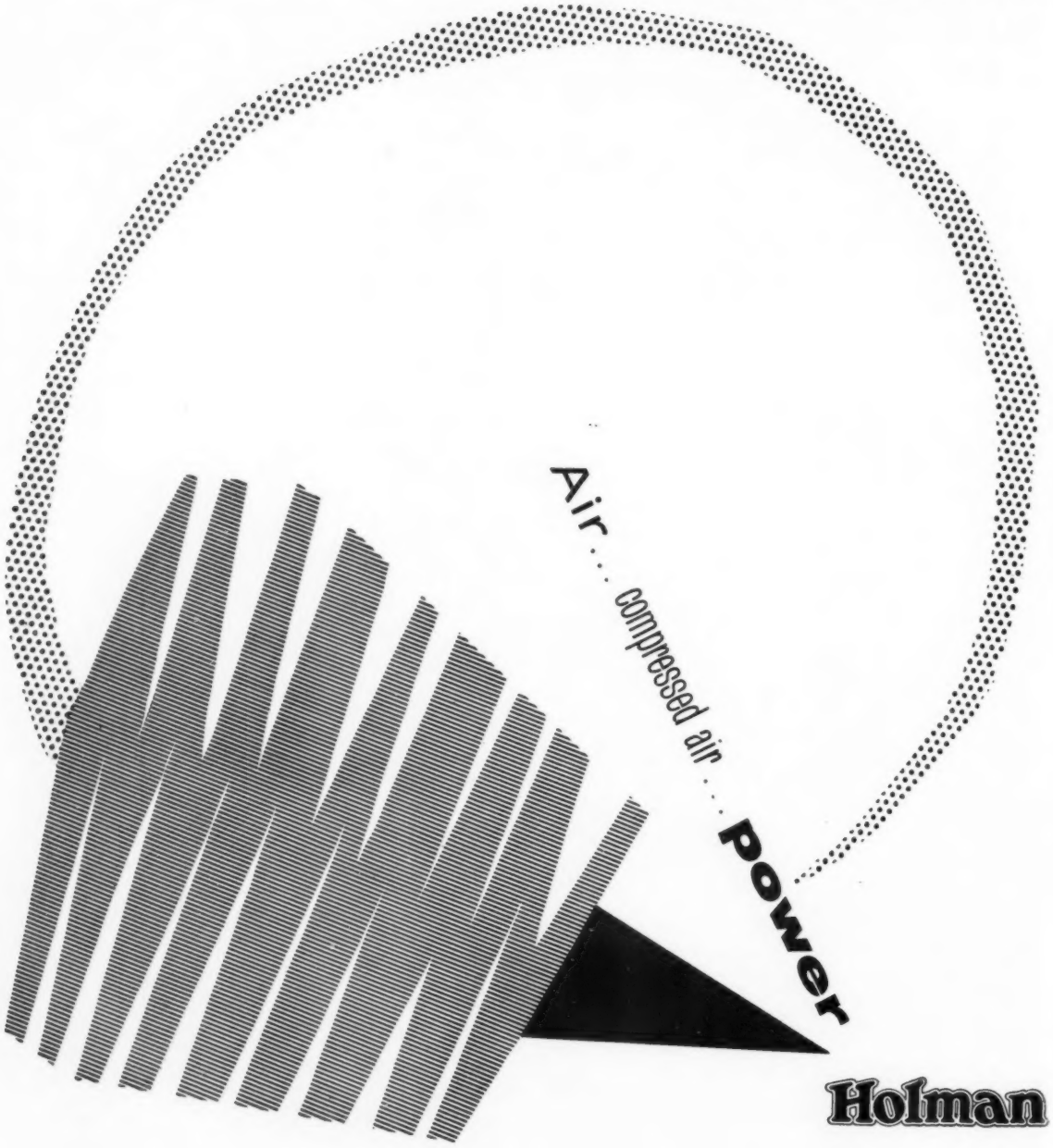

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Abstracts from the World Technical Press

Arimine Dam, Japan

When, after an interval of 10 years caused by the war, Horukiru Electric Power Co. resumed work on this dam which is situated on a tributary of the Joganji River in Honshu Island, they were confronted by the problem of what can be done with the 180,000 cu. yd. of concrete already in place since 1940. The power company wanted to add 100 ft. to the original design height but uncertainties about the old concrete's strength, cracking and weathering, were of concern to the engineers who were in some doubt about the stresses that might concentrate at the old corners with the addition of new concrete. In addition to the proof supplied by core boring that the concrete was strong enough to stand the height increase, the engineers found that fresh unweathered concrete lay close beneath the surface and that most surface cracks did not extend to the foundation. Furthermore, photo-elastic studies showed that overstressing at the edges could be avoided by rounding the old corners. As a result of the engineering investigations, two to eight inches of old concrete were chipped away, the old edges were rounded to 33 ft. radius, and the cracks were grouted. Fifty tons of grout were injected. This 1,650 ft. long concrete gravity structure will, on completion, attain 462 ft. in height. Its present height is about 400 ft. and the storage of water has already begun in the reservoir, 19 years after the start of construction. (*Engineering News-Record*, Vol. 162, No. 21, May 21, 1959, p. 49, 1 f.)

Konkouré Development, Guinea

The enormous bauxite deposits found in the Konkouré Valley, hardly 60 miles away from the excellent harbour of Konakry, prompted EDF to investigate the possibility of harnessing the river to power generation as far back as 1948-49. In spite of abundant precipitation, the regular alternate of dry and rainy seasons unfortunately causes the river flow to vary between 10 cu. m. and 3,000 cu. m. per second. The first problem arising in the study of the project was therefore the regularisation of the flow. The difficulties entailed are best characterised by the fact that 50% of the total yearly discharge flows out in a mere two months, and 80% in four months. The field investigations carried out since 1952 led to the adoption of a two-stage scheme involving a regular yearly production of over 5,000 million kWh under particularly economical conditions. A full description is given in this article of the upstream stage, Souapiti for which studies have now been completed. The outstanding feature of this stage will be a 120 m. high dam of compacted laterite, 25 million cu. m. in volume, which will create a reservoir of 11,000 million cu. m., large enough to ensure a perfectly regular flow from year to year. The dam will be 1,075 m. long and 10 m. wide at the crest, and around 700 m. wide at the base. This is the first time lateritic material is being used on such a large scale in dam construction. There will be two temporary diversion tunnels, 720 and 770 m. long respectively, connecting, one with the spillway, the other with the bottom outlet. On the right bank, four identical intakes will each feed a 171,000 h.p. Francis turbine through shafts excavated in the rock, concrete coated and steel lined. The underground powerhouse, sited under the right-hand abutment of the dam, will shelter four 133 MVA vertical generating sets—one a stand-by—in a hall 118 m. long by 16·5 m. wide; as established by borings, it will be possible to restrict concreting inside the cavern to the ceiling vault. The beams carrying the tracks of the 200 ton twin travelling cranes will be incorporated in the springs of the arch of the vault, thus doing away with the pillars generally encountered in powerhouses. The selection of a special type of crane has enabled the height of the machine-hall between floor and top of the vault to be reduced to 17·2 m. the height below floor being 17·9 m. Quick access to the powerhouse is ensured by a lift running in a vertical shaft which also serves as a cable duct. Vehicles and routine traffic can reach the machine hall by means of a 600 m. long tunnel. The tailrace is an unlined tunnel discharging 340 cu. m. per second into the river, about 6·2 km. downstream from the powerhouse. The downstream end of the tailrace has been designed so as to empty siphon-like below the level of the river, thus retrieving a head of 4·5 m. The project is scheduled to be completed within six years. Work on the dam proper will not extend to more than 150 days in a dry season, while underground work can proceed without interruption. (P. Simon, Chief Engineer EDF, *La Houille Blanche*, Vol. 14, No. 4, July 1959, p. 415, 11 pp., 9 ff.)

Kouilou-Niari Development

The Kouilou River, in the new Republic of Congo, flows into the Atlantic Ocean about 100 miles north of the mouth of the Congo River, and the Kouilou-Niari Project, for which EDF have practically completed planning, is to be sited in the Sounda Gorge, about 50 miles from the mouth of the river. The narrowness and depth of that gorge, together with a particularly good quality of foundation rock lend themselves to the setting up of a very high dam. But the presence of a 335 m. high pass upstream on the left bank of the gorge, has compelled the designers to reduce the height of the dam so as to restrict the normal storage level of the pool to 380 m., and yet this will require the pass to be closed by a dyke so as to raise its crest to about 400 ft. The pool thus created will cover an area of 1,600 sq. km., extending upstream to a distance of 270 km., and its capacity, 35,000 million cu. m., will enable the turbines of the plant to operate under constant discharge which is nearly equal to the annual average flow of the river. Two types of installations are under consideration: (a) six sets totalling 218 MW; (b) eight sets totalling 156 MW, one set being a stand-by in either alternative. The description of the installations refers to the first. With the exception of the dyke closing the pass upstream of the dam, all the main works are concentrated in the area of the first half of the tail stretch of the gorge, and construction work is concentrated to two diversion tunnels and two cofferdams, one upstream, the other downstream of the dam site. The dam itself is to be a thin concrete arch 305 m.

Dam Building

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long at the crest. In view of the constructional difficulties inherent in the concentration of the main works on a restricted space, this dam is to be erected fully independent of the intake structures and of the powerhouse. In its central section, it will rest at El.15 on a massive concrete base in which the penstocks will be embedded down to the upstream wall of the powerhouse. This base also acts as a foundation to the intake towers. On the right bank the dam abuts directly on the rock, while on the left-bank the ski-jump spillway is between the flank of the gorge and the dam. There are three twin intake-towers, i.e., six intakes in all, 13 m. in diameter inside and 108 m. in height. The diameter of the horizontal penstocks, dictated by governing requirements, is no less than 7 m. with 50 mm. thick walls. The horizontal layout was selected with a view to interrupting as little as possible the concreting work required by their erection. In order to reduce to a minimum the length of the powerhouse across the gorge, the six generating sets are arranged staggered in two parallel rows, so that the overall length of the building is only 118 m. at the level of the erection platform, instead of 160 m. with a single row. Each of the vertical Francis turbines, operating from a net head of 101 m. at 125 r.p.m. will pass 240 cu. m. per second. The alternators have a rated capacity of 242 MVA each; their stators are 12 m. in diameter outside and weigh 350 tons each, the rotors are 10 m. in diameter and weigh 600 tons each. It is estimated it will take at least two years to have the dam site dry and ready for construction work to begin; full completion of the scheme will require another four or five years. The volume of rock excavation is estimated at 900,000 cu. m., and the volume of filling material and concrete at 850,000 cu. m. and 750,000 cu. m., respectively. The aggregate weight of the equipment to be manufactured, transported and erected will be about 30,000 tons (J. Francou, Engineer EDF, *La Houille Blanche*, Vol. 14, No. 4, July 1959, p. 426, 13 pp., 8 ff.)

NOTE: An article on the same subject is included in the issue of July 30, 1959, of *Engineering News-Record*, p. 42.

Hinterrhein Hydro-Electric System

Work is now in full swing on all three stages of this project which will harness the uppermost headwaters of the Rhine in the Lei and Madriser Valleys. The underground Ferrera powerhouse with a generating capacity of 185,000 kW and a pumping capacity of 42,000 kW, at the upstream end, the 225,000 kW middle stage at Bärenburg and at the downstream end of the chain, the 235,000 kW plant at Sils, are all progressing at a fast rate. It is expected that construction work will have reached its peak, both as to extent and expenditure, by the end of this year and the beginning of next. Several alterations in the original project are mentioned in the article. At the upstream end of the system, it has been found possible to combine into one, two intakes collecting the waters of the Bregalba torrent and Juferrheim; this increases the length of the Avers-Madris diversion tunnel but considerably shortens the gravity conduits and brings about various constructional and operating advantages, while at the same time, the valley will be much less affected by the intake of water. The greatest alteration is at the Ferrera underground plant where it has been decided to substitute the four Pelton-wheel driven sets contemplated at the outset by three 72,000 kW generating sets with Francis turbines operating from a head of 520 m. This alteration led to the adoption of a pressure, instead of a free-flow, tunnel for the Ferrera-Sufers diversion thus enabling the water from the Sufers reservoir to be conveyed to the Ferrera plant where relay pumps can feed that water into the low-pressure system of the two storage-pumps. Moreover, space is being provided in the powerhouse for a third storage-pump. At the Sufner Törli, the arched gravity dam of the original general project will be replaced by a thin arch structure. At Bärenburg there will be but one pressure shaft 4·2 to 4·0 m. in diameter inside instead of two smaller shafts. At the Sils plant, the major alteration consists in the addition of two single-phase generating sets, 7,000 kW in maximum aggregate capacity whose special purpose will be to feed the Rhaetian railway line. This additional capacity involves an increase in absorbed discharge of 70 up to 73 cu. m. per second without requiring any alteration to the dimensions of the pressure tunnels, surge-tank and pressure shaft. (Chief Engineer L. Kalt, Motor-Colombus AG., Baden, *Schweizerische Beuzeitung*, Vol. 77, No. 26, June 25, 1959, pp. 405/6, 1 map.)

Arch Dam Calculations

The method developed in this article for the analysis of arch dams is based on the distribution of the hydrostatic load between arch and cantilever stresses; it differs from the trial-load method in the fact that the adjustment of displacements at points common to arch and cantilever elements is expressed by means of a system of linear equations. A further advantage of the method over similar procedures is that it makes full use of the trial-load tables published by the Bureau of Reclamation without having to resort to the use of interpolated values. The assumptions made are similar to those introduced into the trial-load method, the sole difference between the two procedures occurring in the analysis of arch elements, which aims at ascertaining the displacements, not at the $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$ points of the semi-arc but at the points of intersection with the cantilever elements. The unit loads also differ, being triangular unit loads which reach their peak value at the point of intersection with but one cantilever element and sink to zero at points of intersection with two adjacent cantilever elements. The number of equations required is equal to the number of intersections between arches and cantilevers. Once we know the deflections due to unit loads on arch and cantilever elements, as well as the initial deflections due to the vertical component of hydrostatic pressure, uplift, temperature, and seismic shocks, it is easy to deduce the corresponding system of linear equations. The authors submit an instance of application in which the unknowns are the values of the loads on the arches. The system of equations can easily be solved by the reduction method since all the equations comprise zero values corresponding to many of the unknowns. New and simplified procedures are introduced into the analysis of the arches, such as foundation deformation at the various phases of the computation or the application of a highly simplified formula

when dealing with the final determination of deflections. This formula does not require the preliminary determination of moments, thrusts or shear stresses at points other than the crown. (J. Laginha Serafim, A. Ferreira da Silveira, J.A. Caldeira Rodrigues and J. de Salvador Fernandes, *Electricidade*, *Revista Técnica Portuguesa*, No. 9, January-March 1959, p. 12, 5 pp., 4 ff., 1 chart.)

Testing Concrete in Large Constructions

In this paper, presented at the Conference on Current Concrete Problems held in Locarno on March 21, 1958, the author deals in detail with the question of preliminary and current tests of concrete. The paper is divided into three main sections: (a) Winning and preparation of aggregates (economic importance of preliminary tests, influence of granular composition on the properties of concrete, upper limit of grain size, apportionment of components, classes of concrete used in hydro-electric developments); (b) Testing methods applied in concrete technology in research on individual causes and effects (petrographic quality of aggregates, granular shape of aggregates, granular composition, quality of cement, cement content of mix, water content of mix, testing fresh concrete as to workability, air content and influence of vibrating; testing hardened concrete by means of sample cubes as to resistance to compression and bending, tensile strength, imperviousness and resistance to frost; conditions affecting hardening); (c) Quantitative determination of the effects resulting from various causes on the quality of concrete (order, and evaluation of laboratory tests, statistical evaluation of test results arrived at in large construction sites); (d) Assessing the safety of a structure from the results of concrete tests; (e) Variations of the individual causes and their effect on the compressive strength of concrete (variations in the quality of basic materials, in mixes, in testing and others, analysing the various fluctuations on site); (f) Observations relating to resistance to frost, and imperviousness. From these observations, it appears that resistance to frost increases with increasing dust content and decreasing sand content, and that samples sensitive to frost are less impervious than frost-resisting samples. The reverse conclusion cannot be drawn, however, since some samples showing an excellent degree of imperviousness offered little resistance to frost. The author admits that his extensive tests supplied no indisputable proof that resistance to frost is due to one cause rather than to another but, as shown previously by Ing. Orth, it is possible that the quality of cement plays an essential part. On the other hand, air entraining agents prove of no avail in this respect, unless air content of the concrete does not reach at least 3%. (Dipl. Ing. H. Bertschinger, *Schweizerische Bauzeitung*, Vol. 77: No. 1, January 1, 1959, p. 8; No. 16, April 16, 1959, p. 233; 18 pp., 30 ff., 6 tables.)

Controlling Concrete Dams

Following an outline of the methods applied in Portugal for the observation of concrete dams for control purposes, this paper deals at length with the results of the investigations carried out on the Cabril Dam, a concrete arch dam 135 m. in height, 292 m. in developed length at the crest, 160 m. in maximum radius, 19 m. thick at the base and 4·5 m. thick at the crest. Some results of measurements of absolute horizontal displacements at points of the downstream face and of vertical displacements at the crest are given and compared with the relative displacements ascertained by means of the pendulum. The main aspects of the behaviour of the structure in respect of the movement of joints and of the evolution of thermal conditions are also reviewed, together with measurements of the water temperature in the reservoir at various depths and in different seasons. A few results of uplift measurements are also shown, analysed and discussed. The procedures applied to the determination of some of the thermic and elastic characteristics of the concrete of the dam, especially diffusibility, coefficient of linear expansion, and Poisson's coefficient, are indicated together with the values obtained for these characteristics. The stresses induced at points 1 m. away from the faces, as well as at mid-thickness of the blocks while the reservoir was being filled are also analysed. A procedure to ascertain the part of the calculated stress due to variations of the thermic conditions is suggested, and the part corresponding to hydrostatic pressure is compared with the measurements obtained in model tests. The displacements and deformations at some points are numerically interpreted, a division being made between the part contributed by hydrostatic pressure and that due to the parameters defining thermic conditions. Comparison of the measurements arrived at with the results of model tests is made whenever possible. The principal conclusions which can be drawn from the results of the observations made at Cabril are stated in their general aspect. A table summariing observsations made on various Portuguese dams together with a specification of the measuring equipment used in each instance is appended. (Manuel Rocha, J. Laginha Serafim, A. F. da Silveira, and Marciano Guerreiro, *Electricidade*, *Revista Técnica Portuguesa*, No. 10, April-June 1959, p. 99, 23 pp., 24 ff.)

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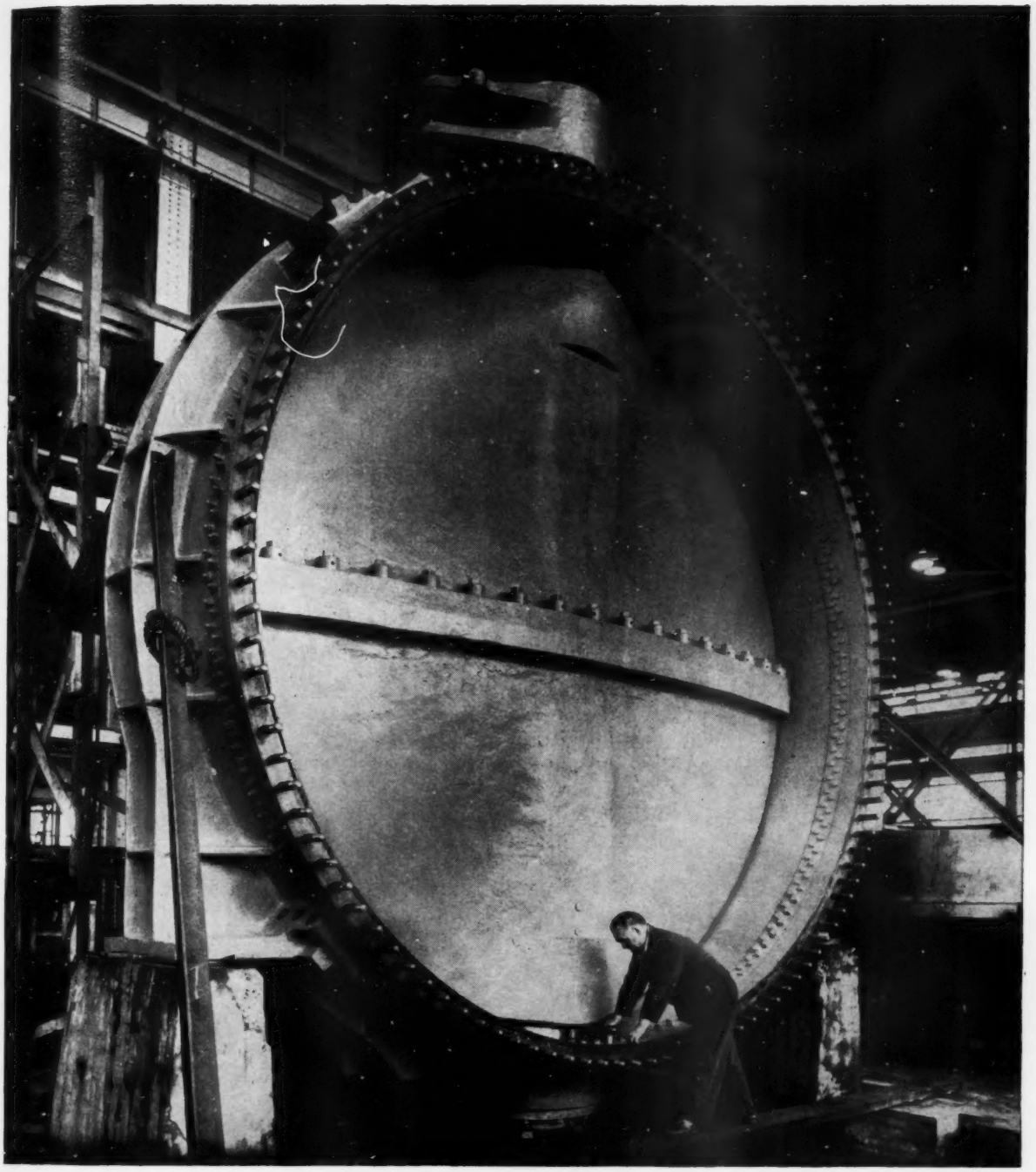

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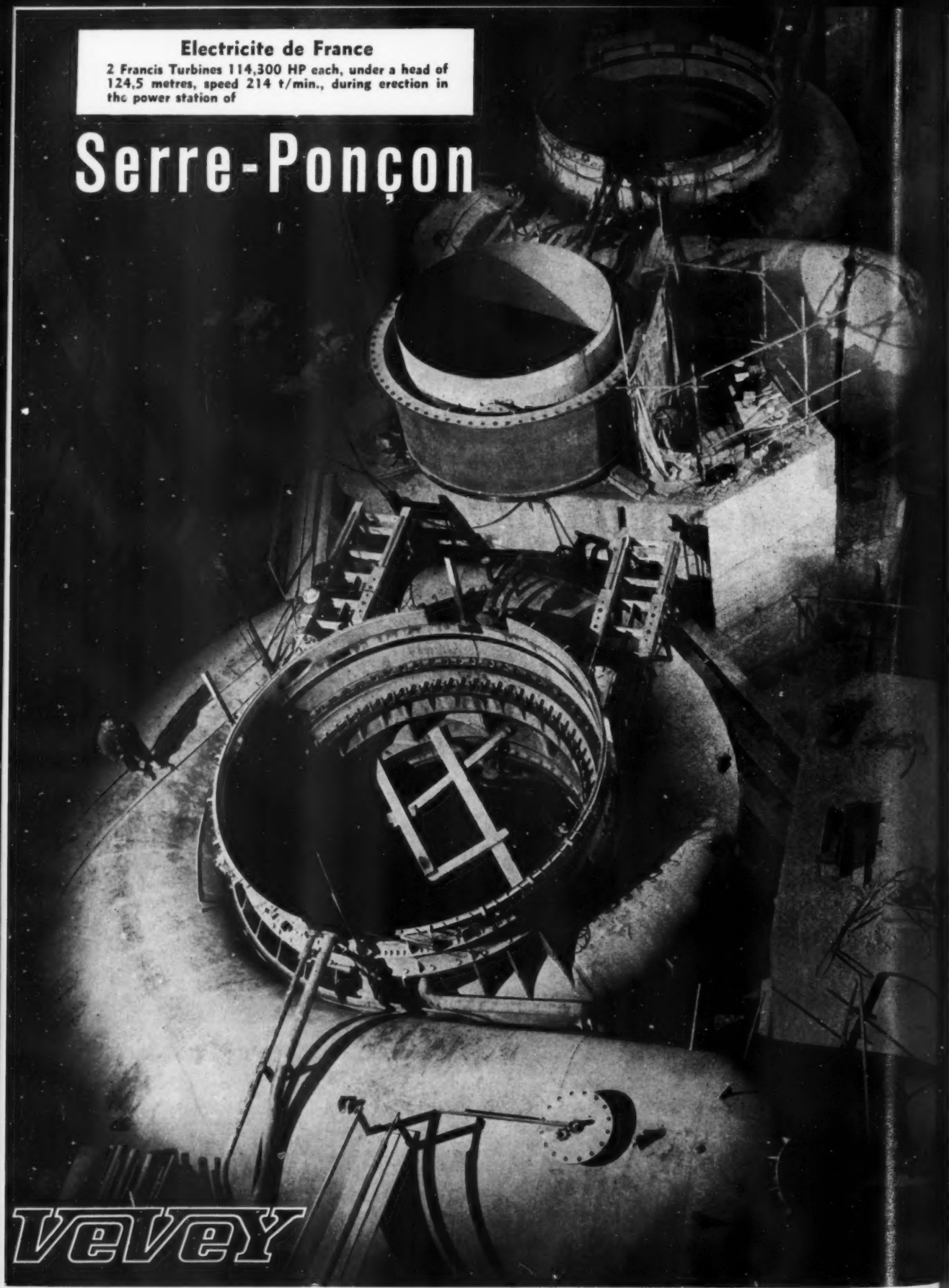

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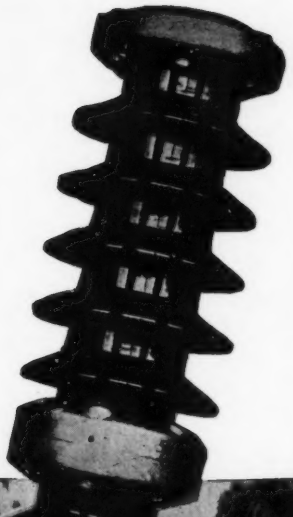

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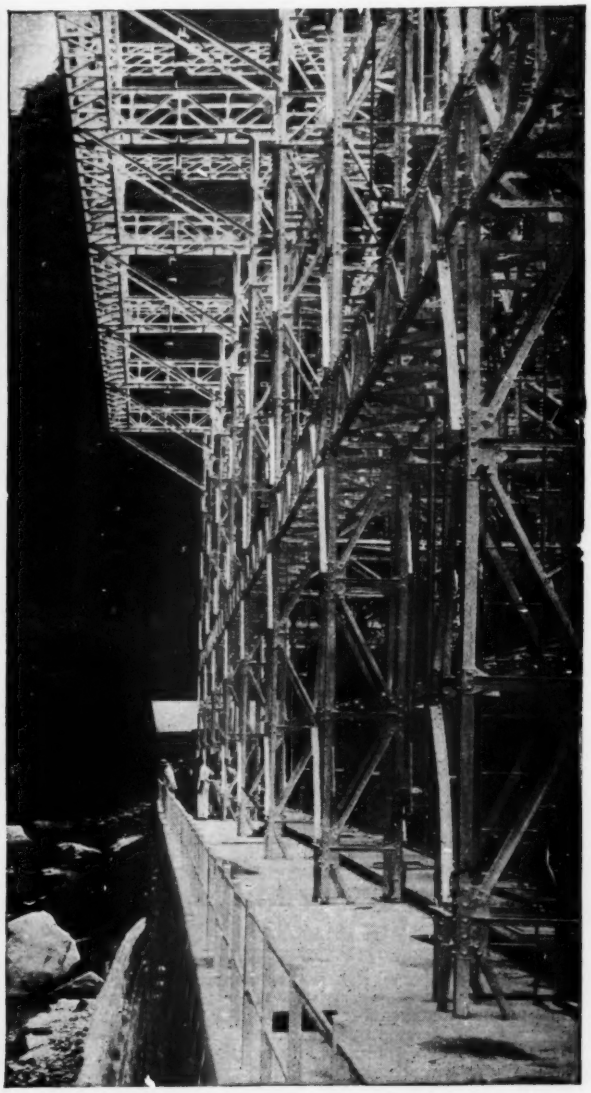

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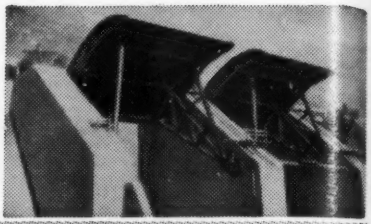

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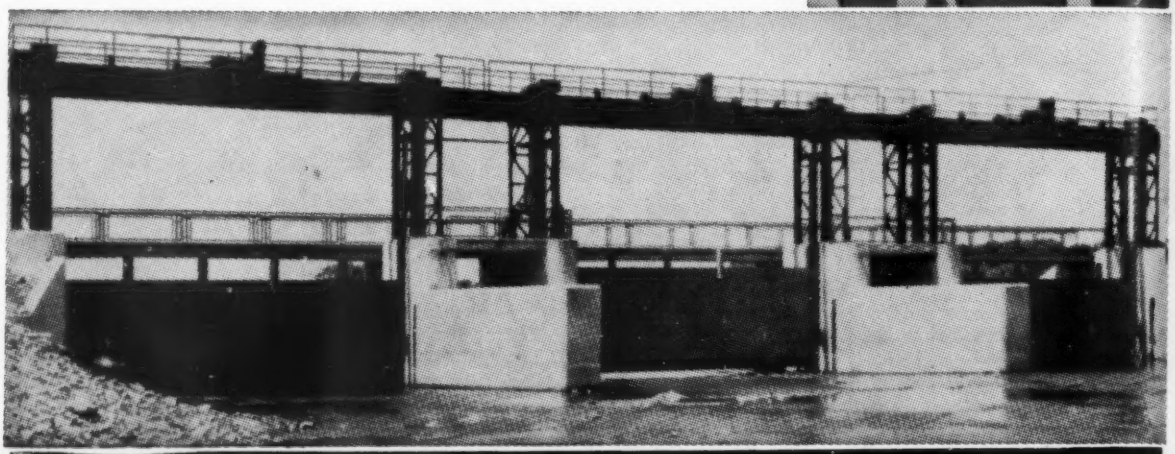

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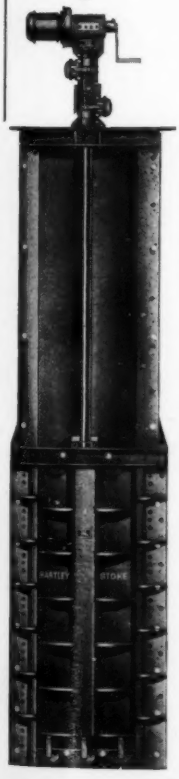

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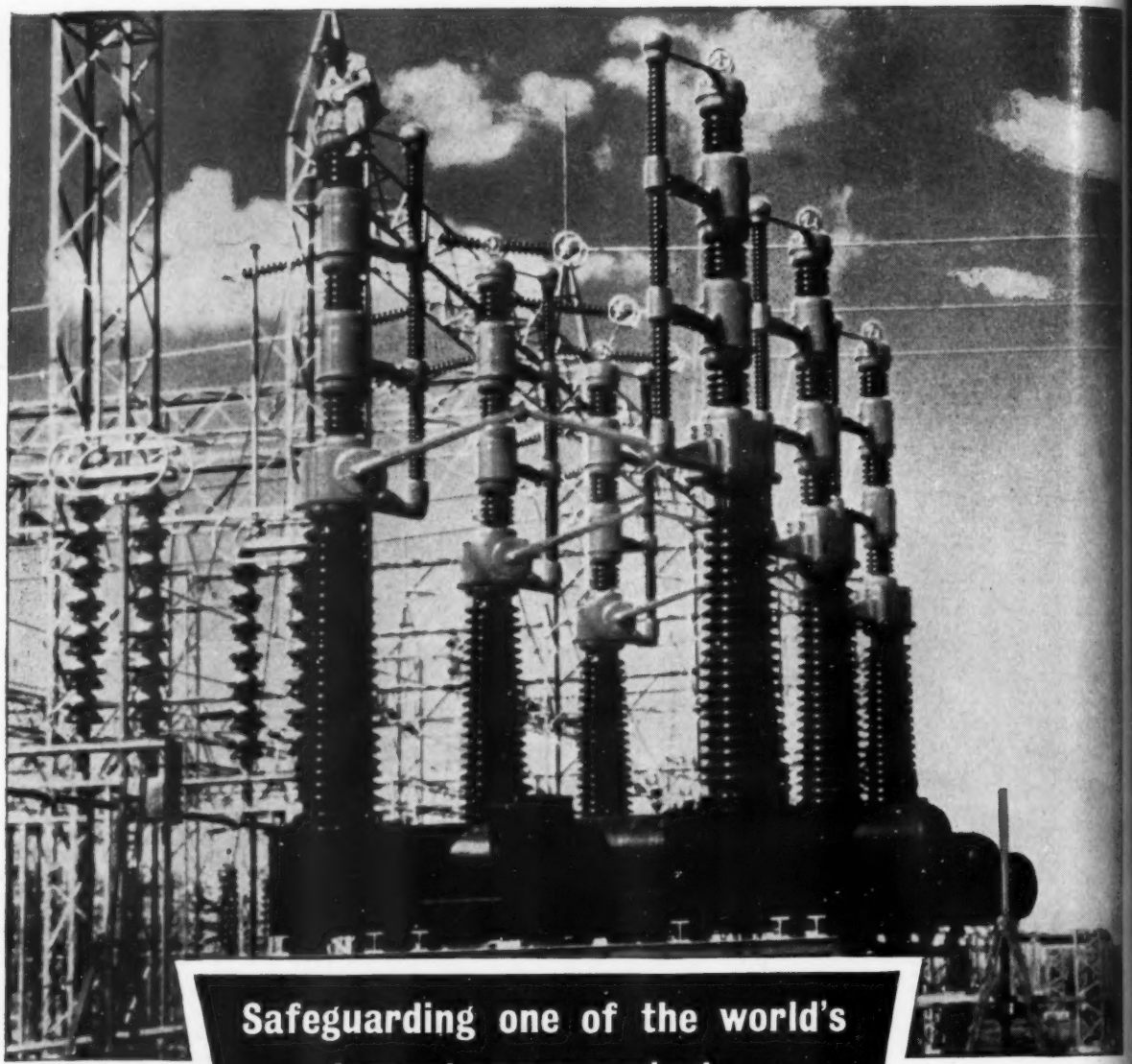

Safeguarding one of the world's largest power projects

The 'ENGLISH ELECTRIC' 330kV 10,000 MVA air-blast circuit-breaker at Hydro Quebec's Bout de L'Ile Terminal Station, which stands at the Montreal end of a 400 miles long 300 kV transmission line from Bersimis Generating Station, one of the world's largest hydro-electric projects. From the terminal station, power is transformed down to 120 kV for distribution throughout the Montreal area. Circuit-breakers of this type are designed for system voltages ranging from 220 kV to 345 kV.

Send for publication SG/250

'ENGLISH ELECTRIC'

switchgear

THE ENGLISH ELECTRIC COMPANY LIMITED, MARCONI HOUSE, STRAND, LONDON, W.C.2.

Switchgear Department, Stafford

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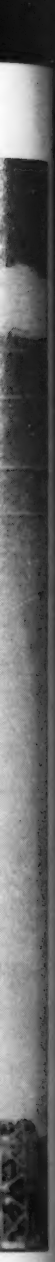
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